CONNECTICUT RIVER FLOOD CONTROL

GARDNER

LOCAL PROTECTION

MAHONEY & GREENWOOD BROOKS
MASSACHUSETTS

DETAILED PROJECT REPORT

(ADVANCE DRAFT)

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS WALTHAM, MASS.

NOVEMBER 1962

U. S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM 54, MASS.

ɔRESS REPLY TO:
.VISION ENGINEER

REFER TO FILE NO.

30 November 1962

NEDGW

SUBJECT: Advance Draft - Detailed Project Report for Gardner Local Protection, Mahoney and Greenwood Brooks, Connecticut River Basin, Gardner, Massachusetts

TO: Chief of Engineers
ATTN: ENGCW-P
Department of the Army
Washington, D. C.

1. In accordance with ER 1165-2-102, there are submitted herewith for review and approval, advance draft copies of the Detailed Project Report "Gardner Local Protection", Mahoney and Greenwood Brooks, Gardner, Massachusetts.

2. Appendix A contains initial letters from the City of Gardner on local cooperation. Upon your approval, formal comment will be requested from the City and also the Commonwealth of Massachusetts and included in final report.

FOR THE DIVISION ENGINEER:

John Wm Leslie

JOHN WM. LESLIE
Chief, Engineering Division

Incl
Draft of Detailed Project Report (7 copies)

CONNECTICUT RIVER FLOOD CONTROL

GARDNER

LOCAL PROTECTION PROJECT

MAHONEY AND GREENWOOD BROOKS, GARDNER, MASSACHUSETTS

DETAILED PROJECT REPORT

U. S. ARMY ENGINEER DIVISION, NEW ENGLAND

CORPS OF ENGINEERS WALTHAM, MASS.

NOVEMBER 1962

TABLE OF CONTENTS

APPENDICES

GARDNER

LOCAL PROTECTION PROJECT

OTTER RIVER - CONNECTICUT RIVER BASIN

GARDNER, MASSACHUSETTS

NOVEMBER 1962

A. PERTINENT DATA

1. Purpose — Overbank flood control of Mahoney and Greenwood Brooks.

2. Location — Mahoney and Greenwood Brooks, City of Gardner, Worcester County, Massachusetts.

3. Type of Improvement — Earth dikes and dams, concrete spillways, channel excavation, arch culvert construction, and removal of channel obstructions.

4. Hydrology

Location	D/A	Maximum Flood Record	Standard Project Flood
Chelsea St.	5.2 sq. mi.	700 c.f.s.	700 c.f.s. (1)
South Main St.	7.7 sq. mi.	1100 c.f.s.	1500 c.f.s. (1)
Winter St.	7.8 sq. mi.	1100 c.f.s.	1500 c.f.s. (1)
Mill St.	9.2 sq. mi.	1280 c.f.s.	1800 c.f.s. (1)

(1) As reduced by flood control storage provided in Wrights Reservoir.

5. Upper Wrights Reservoir — Two 72" x 44" BCCMP with special fabricated flared end sections.

6. Wrights Reservoir — Earth dike 900 feet long, maximum height 14 feet, top elevation 1076, 10 foot top width, 1 vertical on 2 horizontal topsoil and seeded side slopes above normal water surface.
Concrete spillway - 60 feet long, crest elevation 1070, intake elevation 1065, gated draw-down invert elevation 1060, stilling basin elevation 1058.
Channel clearing, excavation and stone paving downstream of the spillway.

7. West Side Wrights Reservoir

Earth and gravel fill dike - 810 feet long, maximum height 11 feet, top elevation 1076, 1 vertical on 2 horizontal, topsoil and seeded side slopes, 16 foot top width, existing earth roadway relocation.

8. Wayside Pond

Earth and gravel fill dike - 320 feet long, maximum height 8 feet, top elevation 1073, 10 foot top width, landside slope varies, waterside slope is 1 vertical on 2 horizontal, topsoil and seeded above normal water surface.
Three 72" x 44" BCCMP (2 with special fabricated flared end sections and 1 with a concrete draw-down inlet.)

9. Mahoney Pond to Chelsea St.

Earth Dikes (2): 390 feet long, maximum height 8 feet, top elevation 1055, upstream of spillway
250 feet long, maximum height 6 feet, top elevation varies, downstream of spillway

Concrete Spillway: 45 feet long, crest elevation 1050.

Channel Excavation and Realignment: 685 feet long, bottom width varies from 45 feet to 10 feet.

10. Quality Pad Company

Lower existing spillway crest 3.7 feet to elevation 1025 and removal of a concrete post which hangs from a bridge over the spillway.

11. South Main St. to Traverse St.

Concrete retaining wall, closed face, crib type, 12.5' high.
Removal of remains of Traverse St. dam.

12. Principal Quantities

Excavation-Unclassified	10,000 C.Y.
Random Earth Fill	6,000 C.Y.
Impervious Earth Fill	12,000 C.Y.
Gravel Bedding	900 C.Y.
Stone Slope Protection	1,700 C.Y.
Concrete Work	1,330 C.Y.
72" x 44" BCCMP	300 L.F.

13. Cost Estimates

First Costs:

Federal	$285,000
Non-Federal	20,000
Total	$305,000 (1)

(1) Exclusive of Pre-authorization costs of $50,000.

Annual Costs:

Federal	$ 8,700
Non-Federal	1,600
Total	$ 10,300

14. Benefits

Average Annual Benefits	$ 15,000
Benefit Cost Ration	1.45 to 1.0

B. PROJECT AUTHORITY

This Detailed Project Report is submitted pursuant to authority contained in Section 205 of the 1948 Flood Control Act; as amended by Section 212 of the Flood Control Act of 1950, Public Law 685, 2nd Session, 84th Congress, adopted 11 July 1956; and Section 205 of Public Law 87-874 of the Flood Control Act of 1962 approved 23 October 1962. Further authority is contained in 1st Indorsement dated 20 March 1961 from the Chief of Engineers to a report dated 26 January 1961 from the Division Engineer, New England Division, Subject: "Reconnaissance Report, Local Protection Project, South Gardner Diversion, Gardner, Massachusetts.

C. SCOPE OF DETAILED PROJECT REPORT

1. SCOPE

This Detailed Project Report reviews the general overbank flood problem at Wrights Reservoir and along Mahoney and Greenwood Brooks into an industrial area in South Gardner, Massachusetts. Gardner is susceptible to floods caused by heavy rains or a combination of heavy rains and melting snow. The flood of record occurred in September 1938. Another major flood occurred in October 1955. These two major floods caused various degrees of damage to several industrial plants and residences along Mahoney and Greenwood Brooks and Foster Branch. This report submits a definite project for overbank flood control by construction of the following improvements: an earth dike, concrete spillway and outlet at the north end of Wrights Reservoir; a flanking dike along the west shore of the reservoir; 2 pipe outlets at Upper Wrights Reservoir; strengthening of the existing outlet structure of Mahoney Brook at Wayside Pond; deepening of Mahoney Brook at critical locations; an earth dam, 3 pipe outlets at the west end of Wayside Pond; a new concrete spillway in the vicinity of the Mahoney Manufacturing Company; a concrete crib wall in the vicinity of South Main Street; and removal of several channel obstructions.

2. TOPOGRAPHIC SURVEYS

A topographic survey of the proposed local protection project on a scale of 1" = 40' and a contour interval of 1 foot was made in September 1961 and July 1962.

3. SUBSURFACE EXPLORATIONS

Geological reconnaissance of the proposed project area has been made. Subsurface explorations were performed during July 1962 and consist of core borings and hand probings. The location and description are shown on Plates 11 and 12. Investigations which are delineated in detail in Appendix D, reveal no particular soil problems in the area of proposed improvement.

4. ECONOMIC INVESTIGATIONS

A detailed flood damage survey was conducted in South Gardner in 1961 to determine the extent of damage that would be experienced in a recurrence of 1938 flood stages. The survey consisted of a field examination of the project area and personal interviews with city and municpal officials and with property owners affected by flooding. Also, an investigation of economic developments and trends was made to project potential future growth and needs relevant to the project.

5. REAL ESTATE STUDIES

Field reconnaissance and conferences with local officials were used as a basis for estimates of real estate costs. Local interests will procure all rights required in land. Present indications are that permanent easements for structures and channel improvement will be adequate. At this time local interests have estimated real estate costs to be approximately $12,000.

6. CONFERENCES WITH LOCAL OFFICIALS

Close liaison has been maintained with city and state officials and other interested parties. Desires of local interests are described in Section O. Officials of local concerns have been contacted and the plan of protection explained. All have expressed a strong desire for the immediate construction and completion of the proposed project. Local interests have supplied firm statements as to their willingness and ability to participate in the proposed improvement. Formal assurances will be furnished by the city and the Commonwealth of Massachusetts prior to completion of final design.

D. PRIOR REPORTS

7. RECONNAISSANCE REPORT

In response to requests from local interests and in compliance with ER 1165-2-102, a reconnaissance report on overbank flooding in Gardner, Massachusetts was made. The report stated that construction of dikes and other channel improvements would relieve the situation. The reconnaissance report indicated that the project was economically feasible and within the scope of Public Law 685. It recommended that the New England Division be authorized to prepare a Detailed Project Report. By 1st Indorsement dated 20 March 1961, the Chief of Engineers authorized preparation of a Detailed Project Report.

E. DESCRIPTION OF AREA

8. GEOGRAPHY

The City of Gardner is located in north-central Massachusetts about 60 miles northwest of Boston and approximately 25 miles north of Worcester, Massachusetts. The project site and area studied within the City of Gardner includes Mahoney and Greenwood Brooks, Wrights Reservoir, Foster Branch, Baker Brook and Ramsdall Pond. A small stream connects Ramsdall Pond to the Otter River which is a principal tributary of the Millers River. The Millers River flows in a generally westerly direction for about 30 miles and eventually finds its terminus in the Connecticut River near the Town of Millers Falls, Massachusetts.

The flood plain along the banks of Mahoney and Greenwood Brooks and Foster Branch contains several chair manufacturing firms, related and other industries. Boston and Maine Railroad spur tracks serve some of the firms. A new supermarket has recently been constructed in the Foster Branch flood plain, upstream from its junction with Mahoney Brook.

The City's flood problem extends for a distance of approximately 1-1/2 miles; from Wrights Reservoir and Wayside Pond to the outlet of Ramsdall Pond, with major damage areas occurring primarily along Mahoney Brook.

9. TOPOGRAPHY

The Otter River and its tributary system, of which Mahoney and Greenwood Brooks are a part, drains a region located within and along the eastern margin of the New England Upland, in central Massachusetts. This is a region of moderate relief, characterized by wide, poorly drained valleys and broad steep-sided hills. The topography, which is to a large degree bedrock controlled, is a result of long-continued normal erosion modified by glacial and post-glacial erosion and deposition. Near the close of the glacial period, vast quantities of outwash were deposited over the region, remnants of which are present in and along the sides of many of the valleys. Above the outwash the slopes are mantled with a variably thick deposit of glacial till. Exposures of bedrock are common to the upper slopes and tops of many of the hills and ridges. Limited outcroppings are also present in and along the bottoms of some of the valleys, where streams have completely removed the glacial debris.

10. SURFICIAL GEOLOGY

In general, the project area is characterized by numerous and extensive swamps, interspersed with low, rounded hills. Mahoney Brook, flowing through the area in a westerly direction, constitutes the principal drainage. In its course through the project, this stream traverses several shallow ponds which have been created by the construction of a number of small dams. Wright's Reservoir, located in the southern part of the project, is also resultant of the damming of Greenwood Brook. For the most part, the streams of the area are flowing upon glacial till, and all have gentle gradients. Overburden, consisting of sandy glacial till, blankets the higher ground in the area, including the low hills. These materials directly overlie bedrock which, to a large degree, controls the topography. In some places these deposits are exposed throughout the valley floor. Elsewhere, near the base of the hills, they disappear beneath scattered remnants of outwash sands and gravels and extensive deposits of swamp muck which occupy the intervening low areas. In some of the developed areas and at the locations of the several existing dikes, these natural materials are overlain by artificial fill. Bedrock, consisting of coarse-grained granite, is exposed at only one location in the project area. This outcropping occurs near the north end of the existing dike which lies along the west side of Wright's Reservoir. Despite the lack of outcrops, indications are that bedrock is present at shallow depths throughout the project area.

11. MAIN STREAM AND TRIBUTARIES

The total watershed area affecting the project site is approximately 7.6 square miles, of which 3 square miles are outside the City of Gardner. The drainage area is made up of three brooks, namely, Foster Branch (2.4 square miles), Greenwood Brook, formerly Nigger Brook (3.2 square miles), and Mahoney Brook, formerly Pew Brook (2 square miles). A description of the characteristics of the brooks is as follows:

Mahoney Brook drains an area east of South Gardner and then is joined by Greenwood Brook and Foster Branch. It rises in a swamp about 2 miles northeast of South Gardner, and flows through rapids, broad valleys, swamps and ponds. There is much valley storage. In its 2 mile stretch through the center of South Gardner it flows through 7 abandoned mill ponds. The profile of Mahoney Brook is characterized by alternately steep and flat reaches. Its total fall is 185 feet. It drains an area of about 2.0 square miles. The principal tributaries of Mahoney Brook are Greenwood Brook and Foster Branch.

Greenwood Brook is a short stream about 1200 feet long which guides the overflow of Wrights Reservoir into Mahoney Brook. It rises in a swamp about 2 miles south of South Gardner and flows northward through a chain of 4 reservoirs to join Mahoney Brook at Mahoney Pond. It drains an area of about 3.2 square miles.

Foster Branch rises in a swamp about 2 miles north of South Gardner and flows southward through 2 ponds and a swamp to join Mahoney Brook about 500 feet upstream from South Main Street Bridge. It drains an area of about 2.4 square miles.

12. STREAM CHARACTERISTICS

a. Mahoney Brook and its tributaries flow through several ponds formed by dams with inadequate spillways and through several culverts and bridges with inadequate openings. For the most part the channels between ponds are narrow and overgrown.

(1) At Wayside Dam there are 2 openings 3 feet by 3.6 feet high (when the boards are completely removed).

(2) At Mahoney Dam there is an outlet channel 10 feet wide with a capacity of 240 c.f.s. before water begins to flow over the top of an existing earth dam. The entrance to this channel is partially obstructed by remains of a gate structure and by a sewer manhole.

(3) At Chelsea Street the waterway under a building is 9.5 feet wide by 5 feet high. Its capacity is 300 c.f.s.

(4) At the dam near Quality Pad Company on the downstream side of Summer Street, the spillway, 10 feet in width, has a capacity of 240 c.f.s. before water begins to overflow the dam.

(5) At South Main Street the stream flows over a rock ledge and the clear span between bridge abutments is only 16.5 feet. Upstream from this constriction the channel bottom has a flat slope and backwater extends upstream beyond Pearson Boulevard. Just downstream from the South Main Street Bridge the abutments of the Boston and Maine Railroad bridge cause the stream to make an abrupt turn. Low steel of this bridge and utility pipes under the highway bridge are about 5 feet above the bed of the stream and obstruct flows in excess of 400 c.f.s.

b. At Wrights Dam, the downstream dam on Greenwood Brook, the existing gate structure has two openings 3.5 feet wide by 5.0 feet high.

(1) The outlet of Upper Reservoir, immediately upstream from Wrights Reservoir, consists of a single 48-inch diameter corrugated metal pipe with a discharge capacity of about 150 c.f.s. which passes under Whitney Street. In addition, there is an existing gated 48-inch diameter corrugated metal pipe. The pipe, under normal conditions, is kept closed.

c. Foster Branch drains the area north of South Gardner. It is about 2.5 miles long and drains several large swamps. It flows over two dams before going underground into an 800 foot box culvert that reduces in size from 13 feet by 8 feet to 10 feet by 6 feet. From this point it flows under a railroad bridge which has a clear opeing of 10 feet by 4 feet and enters Mahoney Brook.

13. MAPS

The problem area of Gardner and its watershed are shown on standard quadrangle sheets of the U. S. Geological Survey (scale 1:31, 680) and on standard quadrangle sheets of the Army Map Service (scale 1:25,000). A basin map of Mahoney, Greenwood and Foster Brooks is shown as Plate No. 1 of this report.

F. CLIMATOLOGY

14. GENERAL

The project area, which for purposes of this report will be referred to as Mahoney Brook Basin, has a modified continental-type of climate. It is generally warm to hot in the summer and moderately cold in the winter. On the average, the precipitation is uniformly distributed throughout the year; but frequently the basin is subjected to short periods of heavy precipitation. The basin lies in the path of the "prevailing westerlies" and cyclonic storms that cross the country from the west or southwest and converge on New England. The area is also exposed to occasional storms that travel up the Atlantic Seaboard, some of which are of tropical origin and of hurricane intensity. These storms have a high potential for flood-producing rainfall, particularly from August to October.

15. TEMPERATURE

The average annual temperature is about 46°F, as indicated by the 12-year record at Birch Hill Dam, located 10 miles northwest of the project site. Recorded extremes are a maximum of 99°F and a miminum of -34°F. The average dates of the first freeze in the fall (32°F or below) and the last freeze in the spring, are September 17 and May 21, respectively. Monthly mean, maximum and minimum temperatures are shown on Table 1.

16. PRECIPITATION

The annual precipitation on the Mahoney Brook Basin is 43-inches as indicated by the 54-year record at Gardner. Annual precipitation has varied from a maximum of 61.01 inches in 1938 to a minimum of

29.60 inches in 1930. Table 1 also summarizes monthly precipitation. The monthly means show that, on the average, the precipitation is uniformly distributed throughout the year. The monthly extremes indicate the existence of wet periods and dry periods.

TABLE 1

MONTHLY TEMPERATURE AND PRECIPITATION

	Temperature (degrees fahrenheit)			Precipitation (inches)		
Station	Birch Hill Dam, Mass.			Gardner, Mass		
Elev. (ft. m.s.l.)	840			1,110		
Years of Record	12			54		
Month	Mean	Max.	Min.	Mean	Max.	Min.
January	24.0	57	-34	3.39	7.35 E	.80
February	26.0	64	-23	3.05	5.40	1.31
March	32.0	70	-17	3.63	9.94	.01
April	45.6	83	10	3.65	7.99	.52
May	55.6	89	20	3.40	7.74	1.07
June	64.3	97	24	3.82	9.21	.52
July	68.5	96	36	3.88	9.21	.76
August	66.6	99	33	3.65	9.41	.45
September	58.8	98	20	3.86	17.31	.20
October	49.4	83	13	3.19	11.83	.07
November	38.9	79	3	3.87	6.92	.67
December	26.9	62	-25	3.36	7.54	.53
ANNUAL	46.4	99	-34	42.75	61.01	29.60

E. One or more days estimated.

17. SNOWFALL

The average annual snowfall on the Mahoney Brook basin is 65.5 inches, based on the record at Gardner, and summarized in Table 2. The water equivalent of the snow cover normally increases through the winter, reaching an average depth of 3 inches in the middle of March, and then diminishes rather rapidly. The maximum snow cover of record had a water equivalent of 8 inches. These figures are based on a 12-year record for the Millers River basin above Birch Hill dam, which includes the Mahoney Brook basin.

TABLE 2

MEAN MONTHLY SNOWFALL
GARDNER, MASSACHUSETTS
(Average Depth in Inches)

Elevation : 1,110 ft. m.s.l.
Period of Record: 18 years

Month	Snowfall
January	18.2
February	16.8
March	11.2
April	4.3
May	0.2
June	0.0
July	0.0
August	0.0
September	0.0
October	0.0
November	4.5
December	10.3
ANNUAL	65.5

18. NOTABLE STORMS

The following tabulation lists notable storms which have occurred at Gardner, Massachusetts.

Date	Total Rainfall at Gardner (inches)	Type of Storm
1936, Mar 16-19	4.60	Cyclone
1938, Sept 17-22	14.52	Hurricane
1955, Oct 14-17	8.30	Cyclone

The "Diane" storm of August 1955 produced 4.50 inches of rainfall at Gardner, but runoff was generally light.

G. RUNOFF AND STREAMFLOW DATA

19. DISCHARGE RECORDS

There are no continuous records of the flow in Mahoney Brook. The U. S. Geological Survey made a survey following the September 1938 flood and estimated that the peak discharge of Mahoney Brook at the middle Mill Street Bridge (drainage area 8.9 square miles) was 1,280 c.f.s.

The U. S. Geological Survey has published records of stages and flow of streams near Mahoney Brook. The records are summarized in Table 3.

TABLE 3

STREAMFLOW RECORDS

Location of Gaging Station	Drainage Area	Period of Record	Discharge (c.f.s.) Mean	Max.	Min.
Tarbell Brook near Winchendon, Mass.	18.2	1916-1960	30.1	2,630	0.1
Millers River near Winchendon, Mass.	83.0	1916-1960	144	8,500	0
Priest Brook near Winchendon, Mass.	19.4	1916-1960	33.5	3,000	0.08
Otter River near Gardner, Mass.	20.0	1916-1917	36.0	192	-
Millers River at South Royalston, Mass.	187	1939-1960	322	4,400	9.3
Ware River near Barre, Mass.	55.0	1946-1960	96.9	1,890	1.2
Ware River at Cold-brook, Mass.	96.8	1928-1960	32*	14,000	4.7

*Large part of flow diverted for water supply purposes.

20. RUNOFF

The average runoff from watersheds near Mahoney Brook ranges from 1.65 to 1.76 c.f.s.m. (cubic feet per second per square mile) for gaging stations with 14 or more years of record. The characteristic variation in monthly runoff is shown on Table 4.

TABLE 4

AVERAGE MONTHLY RUNOFF (c.f.s.m.)

Ware River near Barre, Mass., 1946-1960

Month	Maximum	Minimum	Mean
October	5.00	0.131	0.914
November	4.24	0.602	1.70
December	3.36	0.709	1.87
January	3.71	0.567	2.06
February	4.00	0.876	2.03
March	4.95	1.74	3.03
April	7.29	2.71	4.34
May	3.93	1.19	2.38
June	3.31	0.398	1.15
July	1.65	0.117	0.603
August	3.07	0.068	0.553
September	1.30	0.036	0.401
ANNUAL	2.42	1.11	1.76

H. FLOODS OF RECORD

21. NOTABLE FLOODS

Two notable floods, both aggravated by dam failures, have occurred on Mahoney Brook. These two floods occurred on 19 March 1936 and 21 September 1938.

The flood of 19 March 1936 resulted from a combination of heavy rain and melting snow. The total storm rainfall, March 16-19 at Gardner was 4.60 inches. The associated direct runoff was about 7.2 inches, based on data for adjacent watersheds. (U. S. Geological Survey: W.S.P. 798 - "The Floods of March 1936; Part 1. New England Rivers." 1937. Table 13, p. 352). The dam at the Upper Wrights Reservoir which is utilized as a highway and referred to as Whitney Street was washed out, and increased the inflow into Wrights Reservoir. According to local reports, this caused unusual outflow from Wrights Dam, aggravating downstream flood conditions.

The flood of 21 September 1938 resulted from torrential rains with a total storm rainfall at Gardner of 14.52 inches. The runoff volume, based on discharge records of Priest Brook near Winchendon, was 9.3 inches total runoff in 7 days. Again, the dam and roadway at the Upper Reservoir was washed out. Some part of the flow from Wrights Reservoir was diverted when the dike on the west side of the reservoir was overtopped and washed out, shortcutting some flow into Baker Brook and Ramsdall Pond. The embankment at Wrights Dam was also overtopped but did not fail.

A smaller flood occurred in October 1955, which was reported in the local newspapers. High water data for 1936 and 1938 floods were taken from the publications of the Massachusetts Geodetic Survey: "High Water Data; Flood of March 1936 in Massachusetts," and "High Water Data; Floods of March 1936 and September 1938." Flood elevations are plotted on the profile, Plate 4.

I. FLOOD FREQUENCIES

For the purpose of flood damage evaluations, the flood plain of Mahoney Brook was divided into several reaches. The downstream end of each reach was at a hydraulic control such as a dam, culvert, or restrictive bridge. Discharge frequency curves were drawn for the several reaches. Because there is no continuous record of discharges on Mahoney Brook, use was made of regional flood frequency correlations made by Manuel A. Benson of U. S. Geological Survey. ("Peak Discharge Related to Hydrologic Characteristics in New England," Journal Boston Society of Civil Engineers, Vol. 48, No. 7; January 1961 pp. 48-67.) Comparisons were also made of flood-frequency curves computed from several nearby stream gaging station records by Leo R. Beards method, with values computed by Benson's regression equations. These comparisons indicated that in order to reflect local flood experience adjustments should be made to the regional values. The adjustments computed for Millers River near Winchendon and Priest Brooks near Winchendon were averaged and applied to Mahoney Brook. A stage-frequency curve was computed at each index station from discharge-frequency and stage-discharge relations. The stage-frequency curves were then adjusted on the basis of observed flood heights in order to reflect the effects of debris, ice, and dam failures.

J. STANDARD PROJECT FLOOD

22. STANDARD PROJECT STORM

The standard project storm was determined as prescribed in the Civil Works Engineer Bulletin 52-8, Subject: "Standard Project Flood Determinations." The following tabulation compares maximum depth duration data with that experienced at Gardner during the storm of 17-22 September 1938.

Duration	Maximum Depth in Inches	
Hrs.	Standard Project Storm	Storm of 17-22 September 1938
3	7.4	3.5
6	9.3	5.2
12	10.6	7.1
24	11.8	9.6
48	13.3	12.3
72	13.9	13.6
96	14.2	14.2

23. UNIT HYDROGRAPHS

Unit hydrographs applicable for the standard project flood were determined emperically and were based primarily on unit peaks and lag times. The assumed unit peaks varied from 30 to 100 c.f.s. per square mile. The lag times, in each case representing runoff primarily from land surfaces, varied from 8.0 to 2.6 hours. The size of watershed subdivisions ranged from 1.63 to 0.36 square miles.

24. STANDARD PROJECT FLOOD

Component hydrographs for the standard project flood were determined by applying the standard project storm, less infiltration and other losses to the applicable unit hydrographs. In the Greenwood Brook watershed, the SPF inflow into Upper Reservoir was routed through the existing surcharge storage to obtain the outflow. The dam would be overtopped and failure would occur as it did in the floods of 1936 and 1938. Release of water from the Upper Reservoir, plus local inflow, would cause the existing Wrights Dam and dike on the west side to also fail. The SPF on Mahoney Brook at Wayside Dam would overtop and wash out the existing embankment. Failure of these two dams would release stored waters into the center of Gardner. The channel of Mahoney Brook is inadequate to accomodate the high indeterminate discharges and there would be considerable inundation and damage. The effects of improvements on the SPF are described in Appendix C.

25. SPILLWAY DESIGN FLOOD

The spillway design flood for Wrights Reservoir was based on probable maximum precipitation as given by Hydrometeorological Report No. 33, and minimum loss rates. Depth-duration data (for 10 square miles) were 22.8, 25.2 and 27.3 inches for 6, 12, and 24 hours, respectively. The loss rate was 0.05 inches per hour. The rainfall excess amounts were found to be 2.5 times those of the standard project flood. Assuming the same unit hydrographs, the SPF hydrograph ordinates were multiplied by 2.5 to obtain the spillway design flood ordinates.

K. PROJECT DESIGN FLOOD

The standard project flood was adopted for the project design flood. In general, the improvements will be adequate to pass the standard project flood without damage. The exceptions are the improvements in the vicinity of the Chelsea Street Bridge, the bridges for South Main Street and the Boston and Maine Railroad. The excessive cost of new bridges precludes providing channel capacity to eliminate all damage during the standard project flood. This subject is further discussed in Appendix C, Hydraulic Design.

L. FLOOD DAMAGES AND ECONOMIC DEVELOPMENT

26. EXTENT AND CHARACTER OF FLOODING

Gardner's start as an industrial community began in 1805 with the erection of its first chair factory. It grew slowly at first but then flourished when innovations were introduced which revolutionized the industry. By the early 1900's, plants mushroomed throughout the city and Gardner became known as "The Chair City of the World."

Today, manufacturing still remains an important segment in Gardner's economy, employing 5,040 persons, approximately 70 percent of the employed population, with an annual payroll of $21,030,000. Almost half of all the firms are still engaged in the manufacture of furniture and fixtures. Other industrial activities include, production of toys, sporting and athletic goods, fabricated structural metal products, and vending machines and devices.

Of the 76 manufacturing plants in the city, 11 are located within the flood plain of Pew, now known as Mahoney Brook and Foster Branch in South Gardner. These plants, along with 11 commercial firms, and 42 residences, are susceptible to serious flooding. They employ about 685 people with annual payrolls of $2,700,000. The 1938 flood, the flood of record, inundated approximately 115 acres in South Gardner with depths of up to 7 feet of water. Property valuation in Gardner is presently assessed at $32,800,000.

27. FLOOD DAMAGES

In the record flood of September 1938, 40 residences, 11 industrial firms, and 9 commercial establishments in South Gardner were seriously damaged. Tons of water released from Wrights Reservoir, overflowed Greenwood Brook and flooded buildings to depths 3 to 5 feet over first floor levels. The industrial firms which lie along Mahoney Brook along the reach from Wrights Reservoir to Ramsdall Pond sufferred the heaviest losses. Raw and finished goods stored outside of the plants were washed downstream and much of the stock within the buildings had to be discarded because of water damage.

Two buildings were undermined and several sections of roadway washed out. One of the buildings, a five-story structure, was eventually torn down. From records kept by city officials, and from interviews with plant and shop owners, it is estimated that damages amounted to $350,000.

28. RECURRING AND PREVENTABLE LOSSES

A recurrence of the record flood heights of 1938 would cause damages estimated at $1,050,000 under present economic conditions in South Gardner. Construction of the project would eliminate $896,000 of these losses.

29. AVERAGE ANNUAL LOSSES

Stage loss data were summarized by reaches that have uniform hydraulic characteristics throughout. Recurring losses in each reach were converted to average annual losses by correlation of stage-damage and stage-frequency curves to produce damage-frequency relationships in accordance with standard Corps of Engineers practice. The average annual loss in the reaches of Mahoney Brook downstream from Wrights Reservoir to Mill Street is estimated at $24,400.

30. TRENDS OF DEVELOPMENT

Gardner, Massachusetts has been static, economically, for some time and there is little indication that there will be any noticeable change in the near future. There is no reason to assume a demand for additional space by either industrial or commercial interests in the flood prone area for some time to come, therefor, neither growth nor enhancement was considered in computing benefits. Data on population and economic trends in Gardner are set forth in Appendix B.

31. ESTIMATE OF BENEFITS

Flood damage prevention benefits for the South Gardner project, derived by determining the difference in annual losses under pre-project conditions and those existing after project completion, amount to $15,000. Details of the derivation of benefits are set forth in Appendix B.

M. EXISTING CORPS OF ENGINEERS FLOOD CONTROL PROJECTS

There are no existing Corps of Engineers flood control projects within the City of Gardner or in the immediate vicinity. The nearest flood control dams are located just north of Athol, Massachusetts on the Millers River about 20 miles northwest of Gardner. These two dams, Birch Hill and Tully, which are part of the Connecticut River flood control system, have no beneficial effect upon the project area.

N. IMPROVEMENTS OF FEDERAL AND NON-FEDERAL AGENCIES

There are no Federal or non-Federal improvements within the watershed of Mahoney and Greenwood Brooks.

O. IMPROVEMENTS DESIRED

Several meetings have been held with local interests to determine the feelings of townspeople and local officials towards the proposed plan of flood control. The citizens and manufacturers of the city are desirous of preventing future losses to the various industries along Mahoney Brook and feel that the proposed plan, as outlined herein, will adequately reduce future damage potential. Local interests have expressed a willingness to fully cooperate on this proposal for flood protection. Prints of letters giving the views of local interests are included in Appendix A of this report.

P. FLOOD PROBLEM AND SOLUTIONS CONSIDERED

32. FLOOD PROBLEM

The problem is one of general overflow of Mahoney and Greenwood Brooks into an industrial area of South Gardner. Gardner is susceptible to floods caused by heavy rains or a combination of heavy rains and melting snow. The flood of record occurred during September 1938 at which time 14.52 inches of rain fell within a 5-day period. Another major flood occurred during October 1955 when 8.3 inches of rain fell during a 4-day period. These 2 major floods caused various degrees of damage to several industrial plants and residences along Mahoney and Greenwood Brooks and Foster Branch. Future occurrence of major floods would cause probable dam failures and considerable damage to the area without the protection afforded by the proposed project.

33. SOLUTIONS CONSIDERED

Consideration has been given to all practicable methods of solving the flood problem in Gardner. A plan for diversion of flood waters from Mahoney Brook into Wrights Reservoir thence into Baker Brook was submitted in the Reconnaissance Report. In the light of subsequent detailed investigations this plan of protection was found more costly than the plan as now recommended. Consideration of upstream reservoirs, as suggested by the Soil Conservation Service, revealed insufficient flow reductions for the cost incurred as well as unfavorable economic justification. The most practical and economically justified solution to the problem is the modification of Wrights Dam and Reservoir to provide flood control storage, combined with channel improvements on Mahoney Brook.

Q. PROPOSED IMPROVEMENT

34. GENERAL DESCRIPTION

The project comprises 4 earth dikes, two of which include concrete spillways, construction of a crib wall, installation of several culverts and removal of the remains of a small dam with appurtenant channel improvement. Specifically the plan for flood protection will provide for a new dam, spillway and outlet at the north end of Wrights Reservoir; a flanking dike along the west shore of the reservoir; strengthening of the existing outlet structure at Wayside Pond on Mahoney Brook; 2 pipe outlets at Upper Wrights Reservoir; widening and deepening of Mahoney Brook at critical locations; and the removal of several channel obstructions. Although the plan of improvement for reservoirs are designed safely to pass the Standard Project Flood, in other areas practical limitations prohibit degree of protection provided to something better than record flood. The proposed work has been broken down into 7 work areas, a detailed description of which follows.

35. UPPER WRIGHTS RESERVOIR

Improvements at this location provide for the construction of two B.C.C.M. pipes to supplement existing pipes under the causeway which acts as a dam between Wrights Reservoir and the Upper Reservoir. As indicated on Plate No. 7, the two arched pipes, sized 72" x 44" will have specially fabricated bitiminous coated flared end sections and will be utilized only during periods of flood flows. The lip elevation of the end section pipe is 1073 m.s.l. Existing pipes will carry normal flows from Upper Wrights into Wrights Reservoir.

36. WRIGHTS RESERVOIR

a. Dam and Outlet Structure. Work on Wrights Reservoir, as indicated on Plate 5, includes modifications to the existing dam and outlet, and channel work along Greenwood Brook, from its outlet to High Street. The major feature of work provides for the construction of an earth dam and dike, with a top elevation of 1076 feet above m.s.l. and a top width of 12'-0". The new dam will extend from high ground just east of the Arcadia Plating Company to high ground on the west side. The embankment will consist of an impervious earth fill core with a 6-inch layer of seeded topsoil on the reservoir side. The downstream slope will have a 6-inch layer of seeded topsoil. Both slopes will be 1 vertical on 2 horizontal.

Located approximately 340 feet from the easterly limit of dike is the center line of a new spillway structure aligned so that a straight channel may be constructed from it, to the existing bridge at High Street. A 3 foot by 10 foot box weir with a rim elevation of 1065 m.s.l. will maintain normal water levels. The crest of the 60 foot wide spillway is 5 feet higher at elevation 1070 m.s.l. A 27-inch pipe will carry flows from the weir, through the spillway into the outlet channel. The pipe invert at elevation 1060 m.s.l. will be 1 foot above the level of the spillway outfall, and will permit lowering of the pond. The existing spillway will be utilized to divert flow during construction and later removed and filled.

The spillway abutments walls are designed as gravity type retaining walls, 26 feet high and 2 feet thick, with a 2 foot batter at the base. Both sides of the wall will have 20 foot lengths with an imbedment of 6 feet into the top of the earth dike. The wall will maintain a top elevation of 1076 m.s.l. for a distance of approximately 7'-6" and then will slope at 2 horizontal on 1 vertical to elevation 1070 m.s.l. Spillway design necessitates maintaining the wall at this elevation due to the turbulance of flood waters as they pass out of the stilling basin. The 20 foot stilling basin section will have a 3 foot rectangular baffle on the downstream end, with an invert elevation of 1059 m.s.l. The existing stream to the High Street Bridge will be widened and improved.

b. <u>West Side Flanking Dike</u>. Improvement in this area provides for an earth dike located on the west shore of Wrights Reservoir about one-half mile from the fore-mentioned dam spillway and dike. The dike will prevent the short cutting and diversion of flow from Wrights Reservoir, into Baker Brook and Ramsdall Pond, and thereby negating the effectiveness of the new control and outlet structure at Wrights Dam. The dike, with a top elevation of 1076 m.s.l. will be constructed of compacted earth fill, with 6 inches of seeded topsoil on the reservoir slope and 6 inches of seeded topsoil on the landward slope. The dike, which will terminate into high ground at either end, will have a 12 foot top width and 1 on 2 side slopes.

37. WAYSIDE DAM

Work at this location provides for the strengthening of an existing roadway and control structure and the construction of 3 new pipe outlets which will permit the passage of peak discharges from major floods. Restoration of dam and outlet at this area is an essential component of the project since its present design is not compatible with proposed design flows. The work site is located at the headwaters of Mahoney Brook in the vicinity of Wayside Furniture Company. An embankment section will be constructed on the upstream side of the roadway with a top elevation of 1073 m.s.l., and will tie into high ground near Route No.2 at its southerly end and into a hill on the north side of the pond. (see Plate No. 8) The pipe outlets will be bituminous coated corrugated metal 72" x 44" in diameter and approximately 70 feet long with flared end sections for 2 of the 3 pipes. The third pipe will have a concrete box inlet with stoplogs to permit lowering of the pond when necessary. The flared end sections will have lip elevations of 1069 m.s.l. while the downstream invert of all 3 pipes will be set at elevation 1064.8. The drawdown inlet will have an invert elevation of 1066. Downstream of the pipe the banks will be rock lined, and shaped to channalize flows.

38. CHANNEL IMPROVEMENTS

a. Mahoney Pond to Chelsea Street. Improvements at this location provide for the construction of a concrete spillway, two earth spoil dikes, channel excavation and realignment. The upstream dike, approximately 400 feet in length, will be about 5 feet high with a top elevation of 1055 m.s.l. The new spillway will have a crest elevation of 1050 m.s.l. and have a width of 45 feet. Just downstream from the spillway, the remains of an old concrete dam will be removed. The present channel alignment downstream from the spillway will be altered somewhat to permit smoother channel flows. This new course will require excavation of a knoll on the left bank for a distance of about 100 feet downstream from the spillway. As indicated on Plate No. 9, the cut will be about 13 feet at its deepest location. The channel will have a bottom width varying from 45 feet at the spillway to 10 feet at Chelsea Street. The total length of channel improvement for this reach from spillway to the culvert is 685 feet. Along the left bank upstream of the Synder Construction Company Building, a second dike, 250 feet in length, will be constructed to channalize flood flows and prevent flooding of adjacent buildings. A portion of the Synder Construction Building which is built over the existing channel near the Chelsea Street bridge will be removed.

b. Quality Pad Company. Work at this site provides for the removal of the remains of a spillway at the Quality Pad Company and the removal of a broken concrete post which hangs from a bridge over the spillway. (see Plate 9.) The existing spillway crest is at elevation 1028.7 and the downstream streambed at elevation 1022.4. The existing spillway will be removed to a crest elevation of 1025. Some channel excavation will be necessary upstream of the spillway.

c. Boston and Maine Railroad Bridge. -The complex of the South Main Street bridge and the Boston and Maine Railroad bridge comprises a serious restriction to major flows. Reconstruction and realignment of the bridges are desirable but prohibitive in cost. The only feasible improvement provides for the removal of the dumped material below the railroad bridge and construction of a retaining wall to hold the bank. A concrete pre-cast crib wall approximately 12 feet high and 14 feet in length will be constructed at the northwest abutment of the Boston and Maine Railroad Bridge.

d. Dam at Traverse Street. Work in this location provides for the removal of the partially destroyed Traverse Dam. The dam, located about 200 feet downstream of the Boston and Maine Railroad Bridge, has no present use. Its removal will improve flow conditions at the railroad bridge by lowering the tailwater depth. The dam, composed of mortared masonry blocks, will be removed in such a manner so as not to undermine adjacent structures and buildings.

39. RELOCATION OF UTILITIES

The plan of improvement will not require the relocation of utilities. There are no changes contemplated for existing sewer, water or other drainage lines.

R. MULTIPLE-PURPOSE FEATURE

The Gardner Local Protection Project is designed primarily for flood protection but does include a multi-purpose feature at Wayside Pond. At this location dike construction will insure stability to adjacent banks and prevent the sudden washout of Wayside Dam and Pond, which is currently utilized as a waterfowl habitat. As indicated in Exhibit No. 5 of Appendix A and particularly Paragraph 3 of letter dated 24 November 1961, the proper maintenance of pool levels in this area is essential to this significant wildlife resource.

S. RECREATION DEVELOPMENT

The plan of improvement is solely for flood protection and contains no recreational features. The United States Fish and Wildlife agency, and the Massachusetts Fish and Game Interests have expressed a desire and need for fisherman access into Wrights Reservoir. It has been suggested by these agencies that the new flanking dike at Wrights Reservoir be utilized as an access road. The area is not particularly attractive for public use at this time due to poor access and the absence of publicly owned land. The project design permits inclusion of recreational features at a later date if local interests so desire.

T. ESTIMATES OF FIRST COSTS AND ANNUAL CHARGES

40. GENERAL

Estimates of Federal and non-Federal first costs and annual charges are given in Table 5. These estimates have been prepared on the basis that local interests would bear the cost of the following: removal of building at Snyder Construction Company, and removal of Quality Pad Dam; furnish all lands, water rights and rights-of-way necessary for project construction; and to operate and maintain the project after completion. Unit prices used in estimating costs are based on average bid prices for similar work in the same general area. The prices are based on 1962 price levels and include minor items of work which are not separately detailed in the cost estimates.

41. BASIS OF COST ESTIMATES

Detailed cost estimates have been made upon the basis of a design which would provide an economical and safe structure for the particular site. Estimates of quantities have been made upon the basis of neat outlines of the proposed designs and foundation requirements. Costs were computed as outlined in the Corps of Engineers Engineering Manual 1120-2-104.

42. CONTINGENCIES, ENGINEERING, SUPERVISION AND ADMINISTRATION

To cover contingencies, estimates of construction costs have been increased by 15 percent. The cost of future engineering and design has been taken as 8.6 percent of the construction costs. The cost of supervision and administration has been taken as 7.0 percent of the combined construction and engineering costs.

43. BASIS OF ANNUAL CHARGES

The estimates of annual charges were based on the use of public funds for the total investment over a period of 100 years. Federal annual charges include 2.875 percent of the total investment for interest and 0.179 percent for amortization. Non-Federal annual charges include 4.0 percent of the construction investment with 0.081 percent for amortization plus maintenance costs. The non-Federal interest rate was determined after an investigation had been made of the borrowing power and repayment ability of cities and towns in the Gardner area. Maintenance charges were based on the particular site conditions and previous experience with similar projects.

TABLE 5

ESTIMATES OF FIRST COSTS AND ANNUAL CHARGES

LOCAL PROTECTION, GARDNER, MASSACHUSETTS

FIRST COST

(1962 Base)

FEDERAL

Item	Quantity	Unit	Unit Cost	Amount
Site Preparation	1	Job	L.S.	$ 8,500 (1)
Stream Control	1	Job	L.S.	7,500 (1)
Excavation - Unclassified	10,000	C.Y.	2.00	20,000
Compacted Impervious Fill	12,000	C.Y.	3.00	36,000
Dumped Random Fill	6,000	C.Y.	1.50	9,000
Gravel Bedding	900	C.Y.	5.00	4,500
Stone Slope Protection	1,700	C.Y.	10.00	17,000
Top Soiling and Seeding	10,000	S.Y.	1.00	10,000
Gravel Fill	1,600	C.Y.	3.00	4,800
Concrete Spillway-Wrights Reservoir	800	C.Y.	40.00	32,000
Concrete Spillway-Mahoney Pond	80	C.Y.	40.00	3,200
Concrete Abutment Wall-Wrights Reservoir	370	C.Y.	60.00	22,200
Concrete Abutment Wall-Mahoney Pond	80	C.Y.	60.00	4,800
Concrete Pre-Cast Crib Walls	1	Job	L.S.	1,800
72" x 44" B.C.C.M.P.	1	Job	L.S.	6,400
Gate Structure - Wrights	1	Job	L.S.	5,000
Remove Quality Pad Co. Dam	1	Job	L.S.	3,000
Remove Traverse Street Dam	1	Job	L.S.	2,000
Miscellaneous Items	1	Job	L.S.	1,700
Total				$199,400
Contingencies				30,600
Total Construction Cost				$230,000
Engineering and Design				20,000
				$250,000
Supervision and Administration				22,000
Total Estimated Federal First Cost				$272,000 (2)

(1) Includes costs for work at 7 different locations.

(2) Does not include $50,000 for costs for pre-authorization studies.

 (R 4/26/63)

TABLE 5 (Cont.)

NON-FEDERAL

Item		
72" x 44" B.C.C.M.P. Culverts with Concrete Inlet at Wayside Pond		$ 11,000
Removal Synder Building		8,000
Lands and Damages		12,000
Total Estimated Non-Federal First Cost		$ 31,000
Total Estimated Project First Cost		$303,000

ANNUAL CHARGES

FEDERAL

Interest (.02875 x $272,000)	$7,810	
Amortization (.00179 x $272,000)	490	
Total Federal Annual Charges		$ 8,300

NON-FEDERAL

Interest (.02875 x $31,000)	$ 890	
Amortization (.00179 x $31,000)	55	
Maintenance	455	
Tax Loss	200	
Major Replacements	600	
Total Non-Federal Annual Charges		$ 2,200
Total Annual Charges		$ 10,500

Benefit-Cost Ratio = $\frac{\$15,000}{10,500}$ = 1.43 to 1.0

(R 4/26/63)

U. ESTIMATES OF BENEFITS

Average annual flood damage prevention benefits, taken as the difference between average annual losses under existing conditions and those losses remaining after construction, amount to \$15,000. It is not anticipated that the project will result in any increased utilization or enhancement benefits. Details of the derivations of benefits are set forth in Appendix B.

V. COMPARISON OF BENEFITS AND COSTS

Average annual benefits for the Gardner Local Protection Project are estimated at \$15,000 and average annual costs are estimated at \$10,300. The resulting ratio of benefits to cost is 1.45 to 1.0.

W. PROJECT FORMULATION AND ECONOMIC JUSTIFICATION

The Division Engineer finds that past floods have caused substantial damages to the lands and existing structures along Mahoney and Greenwood Brooks in Gardner. He concludes that a plan of improvement consisting of a new dam, spillway and outlet at the north end of Wrights Reservoir; a flanking dike along the west shore of the reservoir; strengthening of the existing outlet structure at Mahoney Brook; 2 pipe outlets at Upper Wrights Reservoir; widening and deepening of Mahoney Brook at critical locations; construction of a crib wall, and the removal of several channel obstructions would provide the maximum practicable degree of protection for this area against future flooding. Project formulation resolved itself into one plan of protection which had not only economic justification but also afforded construction feasibility and compatibility with existing improvements in the area. Protection can be provided most suitably by the plan as submitted herein for approval.

The major feature considered in the formulation of various alternate plans of protection concerned improvements in the vicinity of the existing outlet to Wrights Reservoir. Failure at this critical area could result in the sudden release of large volumes of water which could scour out several establishments along Mahoney Brook, incurring extensive damage and possible loss of life. The strengthening and reconstruction of the Wrights Reservoir Dam, and the outlet structure at Wayside Pond will prevent such an occurrence. Other improvements are largely influenced by existing structures that economically and practically limit the scope of feasible work.

Total project costs of the recommended plan are estimated at \$305,000 exclusive of pre-authorization costs of \$50,000, of which \$285,000 represents the Federal share and \$20,000 the non-Federal share. The plan of protection will yield average annual benefits of \$15,000 as against annual costs of \$10,300, producing a benefit-to-cost ratio of 1.45 to 1.0.

During the development of the recommended plan of improvement, consideration was given to the protection of the Bents Brothers Company property situated downstream of Bents Pond. The construction of a new concrete flume, and a bascule gate with appurtenances, replacing existing sluice gates that are owned and operated by Bents Company was included in the plan of protection. These protective measures would have incurred an increase in project cost of $120,000, $70,000 non-Federal and $50,000 Federal. However, plant officials, during subsequent requested conferences, questioned the need for protective works in this general area. They indicated further that in the past, warning and extensive sand-bagging of openings in their buildings located along Winter Street, "holds water in plant buildings to minimum elevations which cause little damage." Plant officials stated, both the 1936 and 1938 flood effects at Bent Brothers Company were due to surges from failures of Upper Wrights Pond; and rectification of this condition would be all the protection warranted for Bent Brothers. As a result of our meetings senior plant officials requested revision downward of previously provided damage data. In light of this revised information the construction of protective measures was then found unjustified.

In letters to the Mayor of Gardner, it was stated that from a hydrologic viewpoint, this firm could be subjected to flood conditions. The proposed protective works for Gardner are independent of any improvement at Bent Brothers Company.

X. SCHEDULES FOR DESIGN AND CONSTRUCTION

44. DESIGN

It is estimated that preparation of contract plans and specifications for the project can be completed in 6 months after approval of this report. The estimated cost is $20,000.

45. CONSTRUCTION

Construction of the project can be accomplished under a single contract to be completed in a 10-month period. Funds for the construction of the project would be requested upon evaluation of bids received.

Y. OPERATION AND MAINTENANCE

Maintenance of this project will be the responsibility of local interests. Periodic inspections will be made to assure that adequate maintenance is performed in accordance with regulations prescribed by the Secretary of the Army. It is estimated that maintenance of the project will cost local interests $430 annually. An operation and maintenance manual will be provided to the City of Gardner upon completion of the project.

Z. LOCAL COOPERATION

In accordance with Public Law 87-874, local interests would be required to provide without cost to the United States, all lands, easements and rights-of-way necessary for the construction and operation of the project; hold and save the United States free from damages due to construction work; and maintain and operate all the works after completion in accordance with regulations prescribed by the Secretary of the Army. The responsibility for furnishing disposal areas for excavated materials not used in the project and for the modification and removal of bridges and relocation of utilities would rest with local interests under the requirements of lands, easements, and rights-of-way. Removal of portion of the Snyder Construction Company building which now spans Mahoney Brook, presenting an obstruction to major flood flows, will be accomplished and paid for by local interests. Local interests would also be required to furnish the added assurance that they would contribute to the United States all necessary funds over and above the Federal cost limitation of $1,000,000. State and town officials have indicated a willingness to fulfill conditions of local cooperation. A letter from the Mayor of Gardner which constitutes preliminary assurances is included in Appendix C of this report.

In the areas where easement takings are not required for construction of project features, the City of Gardner will establish ordinances to prevent further encroachment in the natural flood plain of the stream. This will be accomplished by the establishment of a flood way which is defined as the area within 25 feet of the centerline of the stream. The restrictions in the flood way, which is reserved for the passage of the design flood, will include the following: (1) no new construction of any type will be permitted, and (2) existing hazardous structures will be removed when obsolete.

In addition, and as a condition of the project, local interests will establish channel lines within the limits of the improvement satisfactory to the Corps of Engineers. These channel lines which will be located beyond the line which establishes the floodway area noted above, will insure that any future building or development will be erected at a safe elevation to minimize future damages from a major flood. The elevation of the design flood is considered to be a safe elevation for this purpose. Therefore, any future development between the floodway and the channel lines can be permitted provided that the buildings are "flood-proofed" to at least the design flood elevation.

AA. COORDINATION WITH OTHER AGENCIES

Plans for local protective works in Gardner have been reviewed by officials of the City of Gardner and the Commonwealth of Massachusetts. The project has no effect on hydro-electric power generation, recreation, pollution abatement, fish migration or other collateral water resource uses.

BB. CONCLUSIONS

Investigations and studies for the local protection project covered by this report lead to the following conclusions:

a. The City of Gardner faces the threat of heavy damages in future floods. A recurrence of the flood of record would cause damages of $980,000.

b. The desires of local interests are for the highest form of protection that can be provided to their principal industries and thereby secure the economic base of the City.

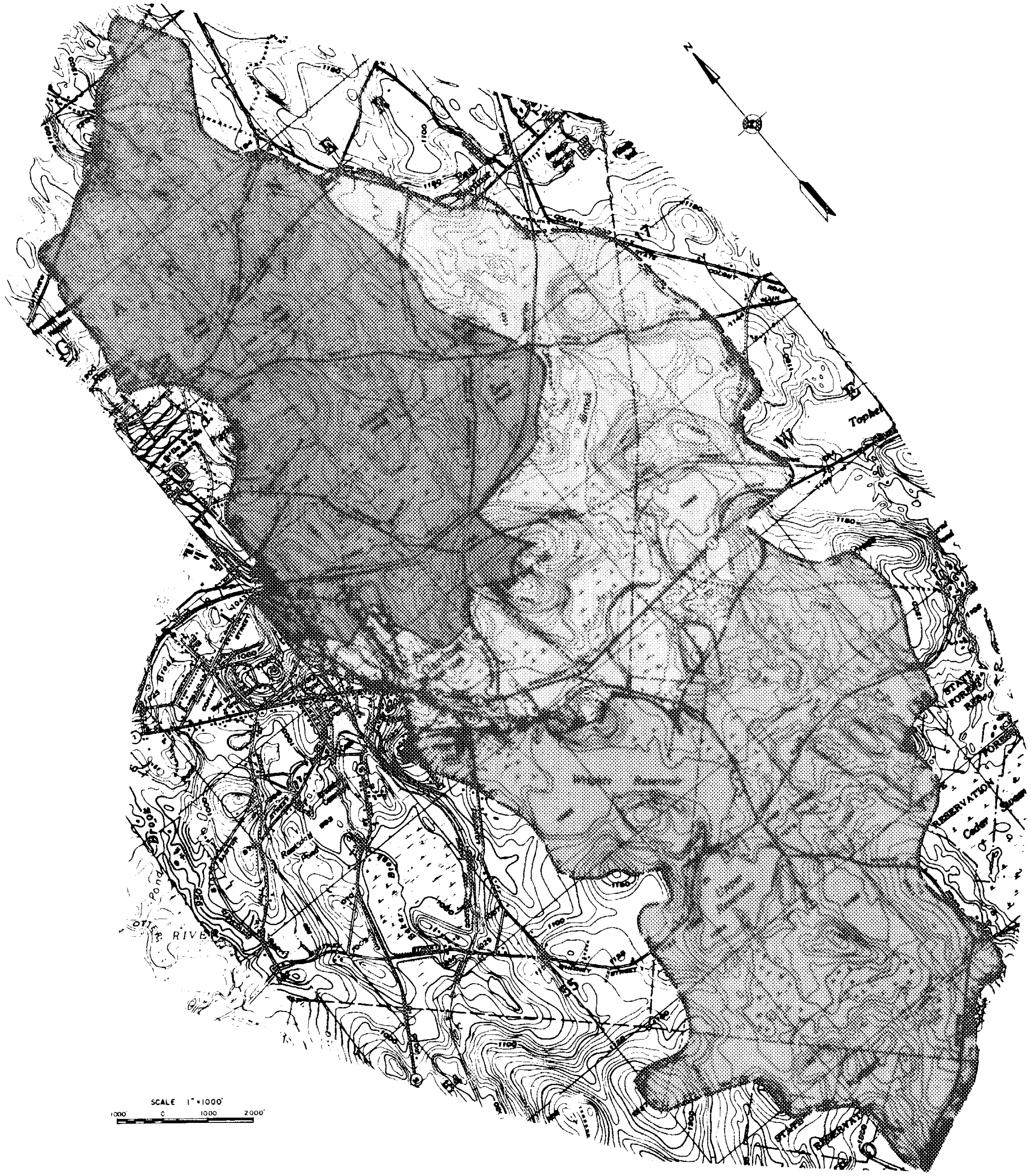
c. Protection can be provided most suitably by the proposed plan at a total estimated Federal first cost of $285,000.

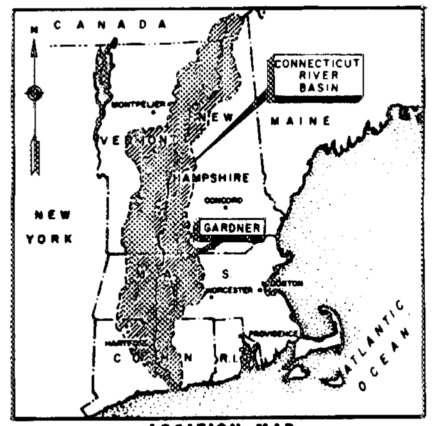
d. The project is economically justified by the ratio of annual benefits to annual costs of 1.45 to 1.0.

e. The threat of recurring damaging floods makes it desirable to construct the project as soon as possible.

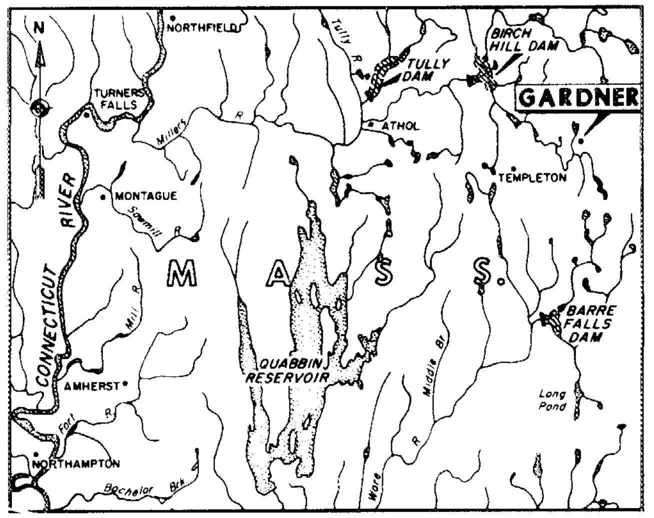
CC. RECOMMENDATIONS

It is recommended that the project, as submitted in this report, be authorized by the Chief of Engineers under the provisions of the Flood Control Act of 1948, as amended, and that additional funds be allotted in the amount of $20,000 for preparation of plans and specifications. Funds for construction will be requested upon receipt and analysis of bids for construction.





LOCATION MAP
SCALE IN MILES



VICINITY MAP
SCALE IN MILES
4 0 4 8

REVISION	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

DR. BY EWH | TR. BY MWB | CK. BY JTT

CONNECTICUT RIVER FLOOD CONTROL
GARDNER MASS.
LOCAL PROTECTION PROJECT
MAHONEY BROOK MASS.

PROJECT ENGINEER
CHIEF, SECTION
SUBMITTED BY
CHIEF, PLANS & RPTS BRANCH
APPROVED
CHIEF ENGINEERING DIV.
DATE OCT. 1962

SCALE AS SHOWN | SPEC. NO CIV ENG.-19-016-
DRAWING NUMBER
CT-1-5863
SHEET 1

PLATE NO. 1

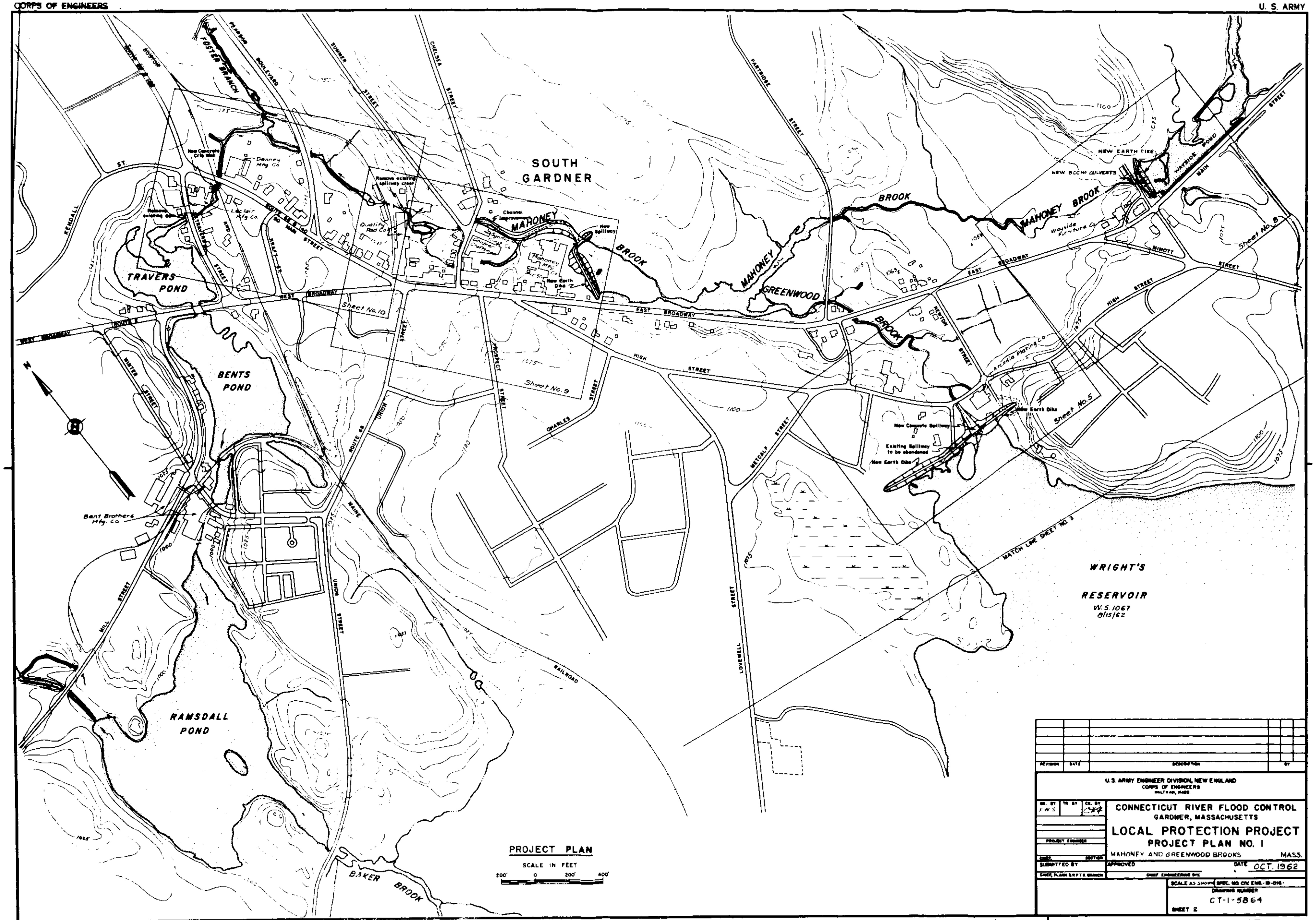

SOUTH GARDNER
MAHONEY BROOK
GREENWOOD BROOK
FOSTER BRANCH
TRAVERS POND
BENTS POND
RAMSDALL POND
WRIGHT'S RESERVOIR
W.S. 1067 8/15/62
BAKER BROOK
EAST BROADWAY
WEST BROADWAY
HIGH STREET
PROSPECT STREET
UNION STREET
LOVEWELL STREET
METCALF STREET
WINTER STREET
MILL STREET
CHARLES STREET
RAILROAD
MATCH LINE SHEET NO. 3
Sheet No. 5
Sheet No. 8
Sheet No. 9
Sheet No. 10
New Concrete Crib Wall
Denney Mfg. Co.
Leclair Mfg. Co.
Remove existing dam
Remove existing spillway crest
Quality Pad Co.
Channel Improvement
Mahoney Mfg. Co.
New Spillway
New Earth Dike
Wayside Furniture Co.
NEW EARTH DIKE
NEW BCCMP CULVERTS
Arcadia Plating Co.
New Concrete Spillway
Existing Spillway to be abandoned
Bent Brothers Mfg. Co.
PROJECT PLAN
SCALE IN FEET
200' 0 200' 400'
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
CONNECTICUT RIVER FLOOD CONTROL
GARDNER, MASSACHUSETTS
LOCAL PROTECTION PROJECT
PROJECT PLAN NO. 1
MAHONEY AND GREENWOOD BROOKS MASS.
DATE OCT. 1962
CT-1-5864
SHEET 2

PLATE NO. 2

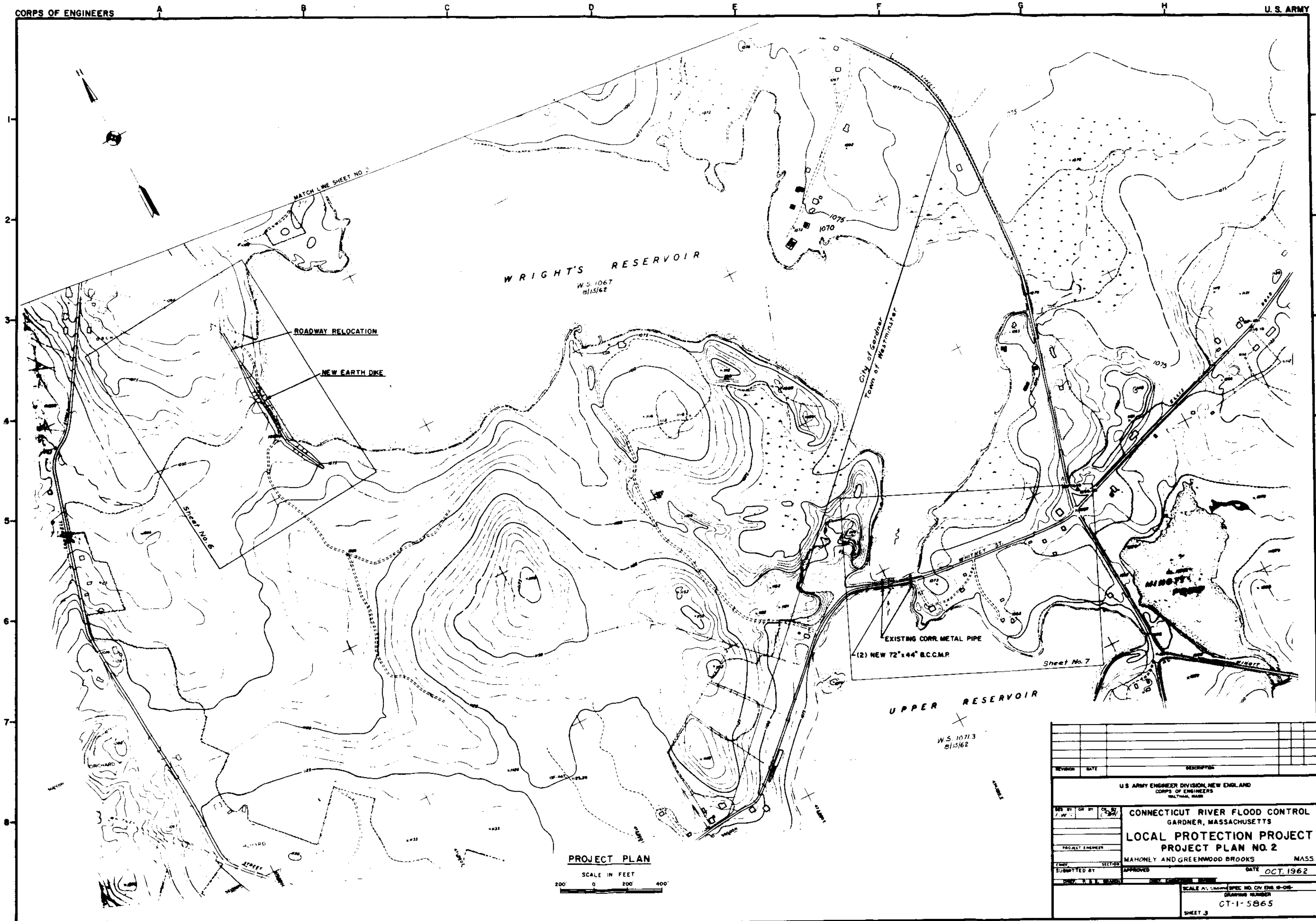
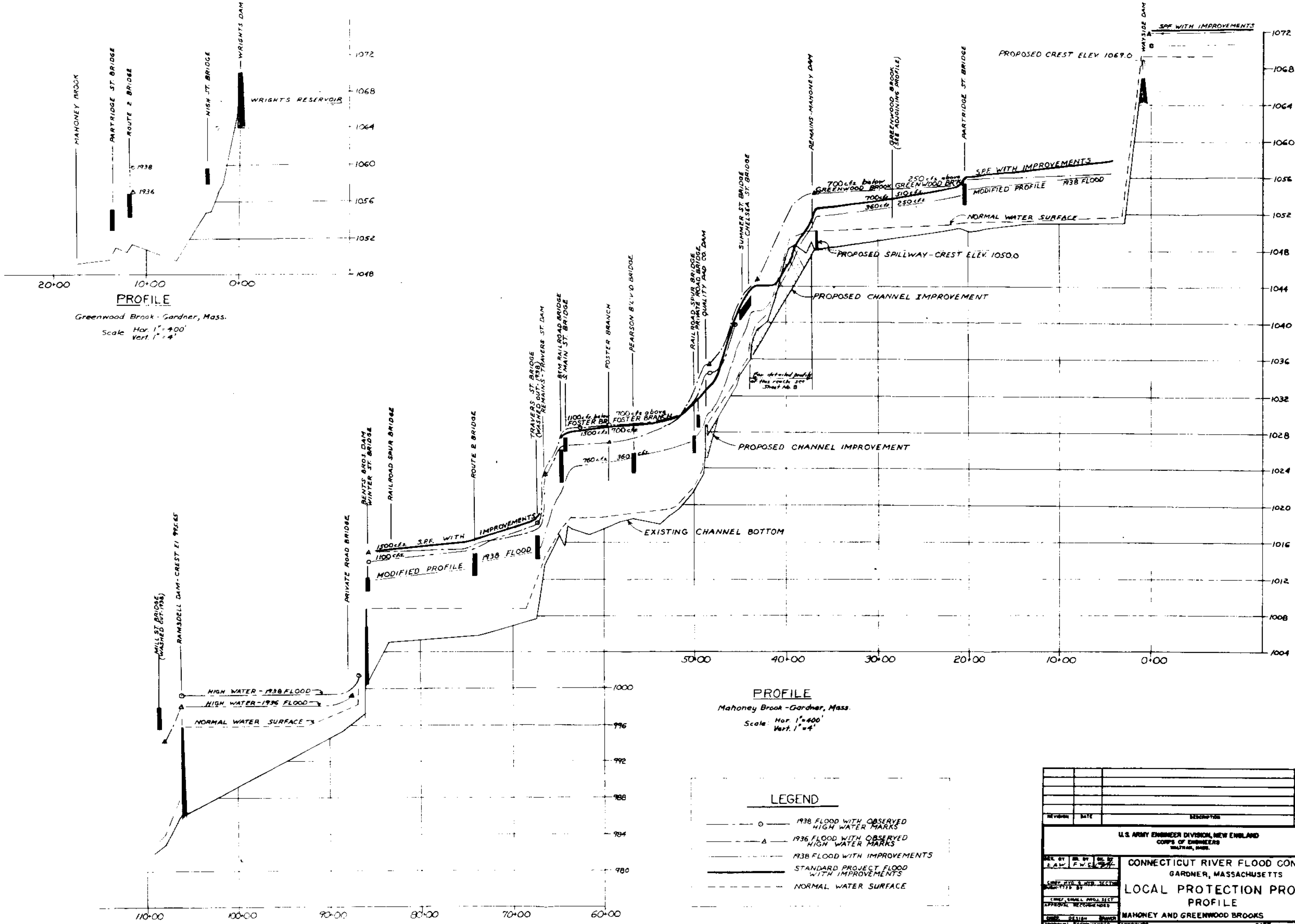

CORPS OF ENGINEERS
U. S. ARMY
MATCH LINE SHEET NO. 2
WRIGHT'S RESERVOIR
W.S. 1067
8/15/62
ROADWAY RELOCATION
NEW EARTH DIKE
Sheet No. 6
City of Gardner
Town of Westminster
1075
1070
WHITNEY ST.
EXISTING CORR. METAL PIPE
(2) NEW 72"x44" B.C.C.M.P.
Sheet No. 7
UPPER RESERVOIR
W.S. 1071.3
8/15/62
ORCHARD
STREET
PROJECT PLAN
SCALE IN FEET
200' 0 200' 400'
REVISION
DATE
DESCRIPTION
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
CONNECTICUT RIVER FLOOD CONTROL
GARDNER, MASSACHUSETTS
LOCAL PROTECTION PROJECT
PROJECT PLAN NO. 2
MAHONEY AND GREENWOOD BROOKS
MASS.
PROJECT ENGINEER
SUBMITTED BY
APPROVED
DATE OCT. 1962
SCALE AS SHOWN
DRAWING NUMBER
CT-1-5865
SHEET 3

PLATE NO. 3



U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

CONNECTICUT RIVER FLOOD CONTROL
GARDNER, MASSACHUSETTS

LOCAL PROTECTION PROJECT
PROFILE
MAHONEY AND GREENWOOD BROOKS MASS.

DATE OCT. 1962

SCALE AS SHOWN

DRAWING NUMBER CT-1-5866

SHEET 4

CORPS OF ENGINEERS

U. S. ARMY

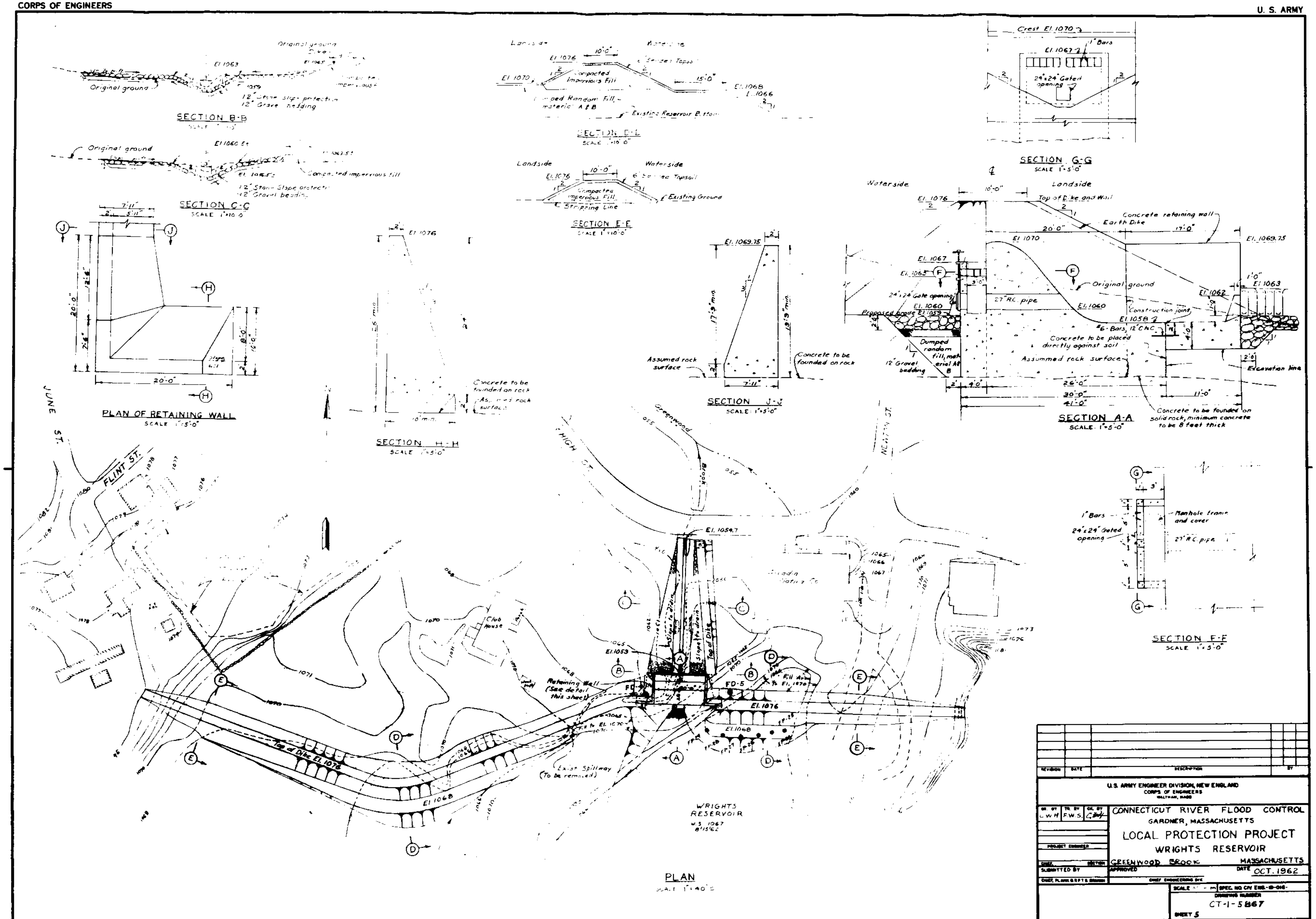
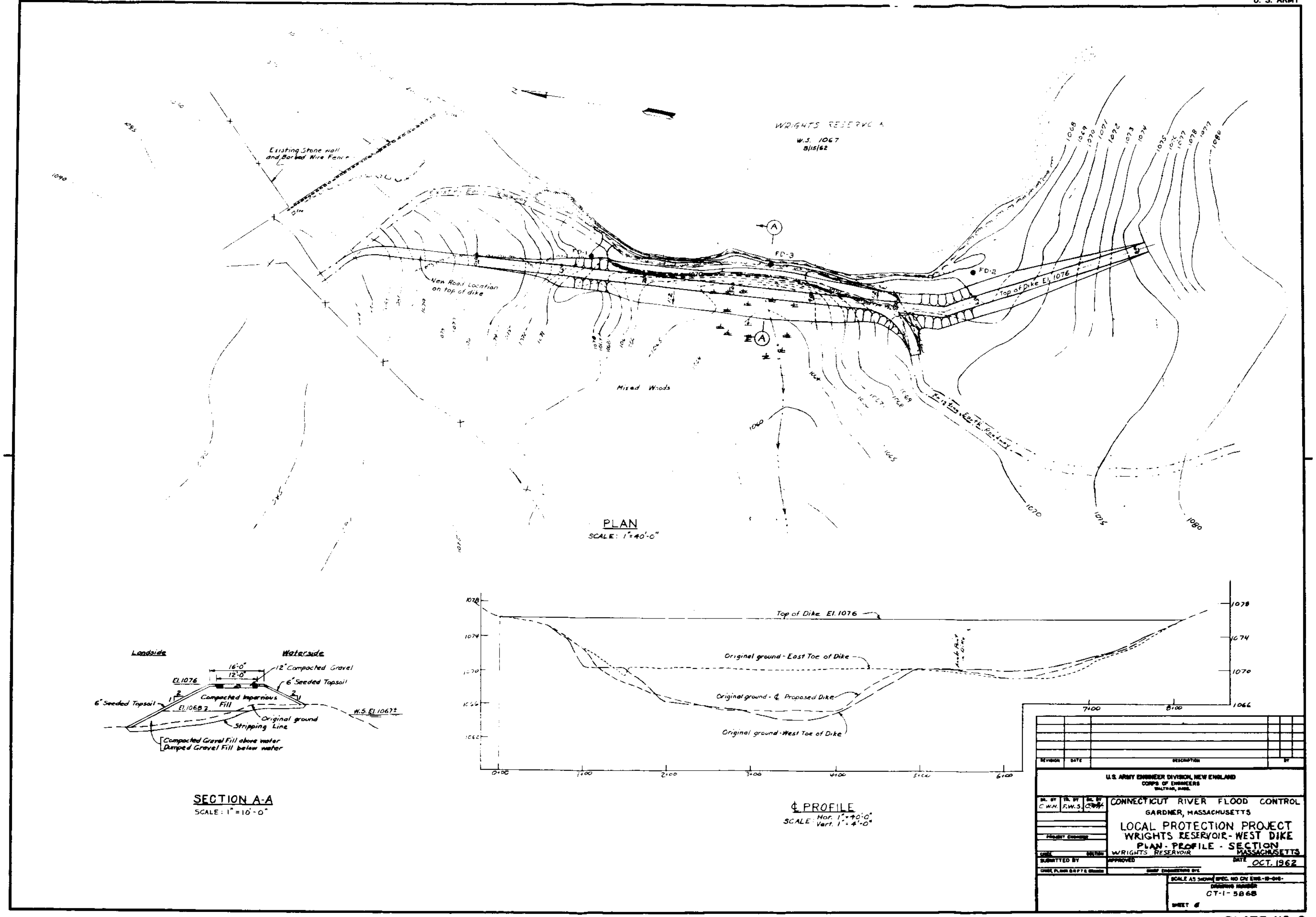
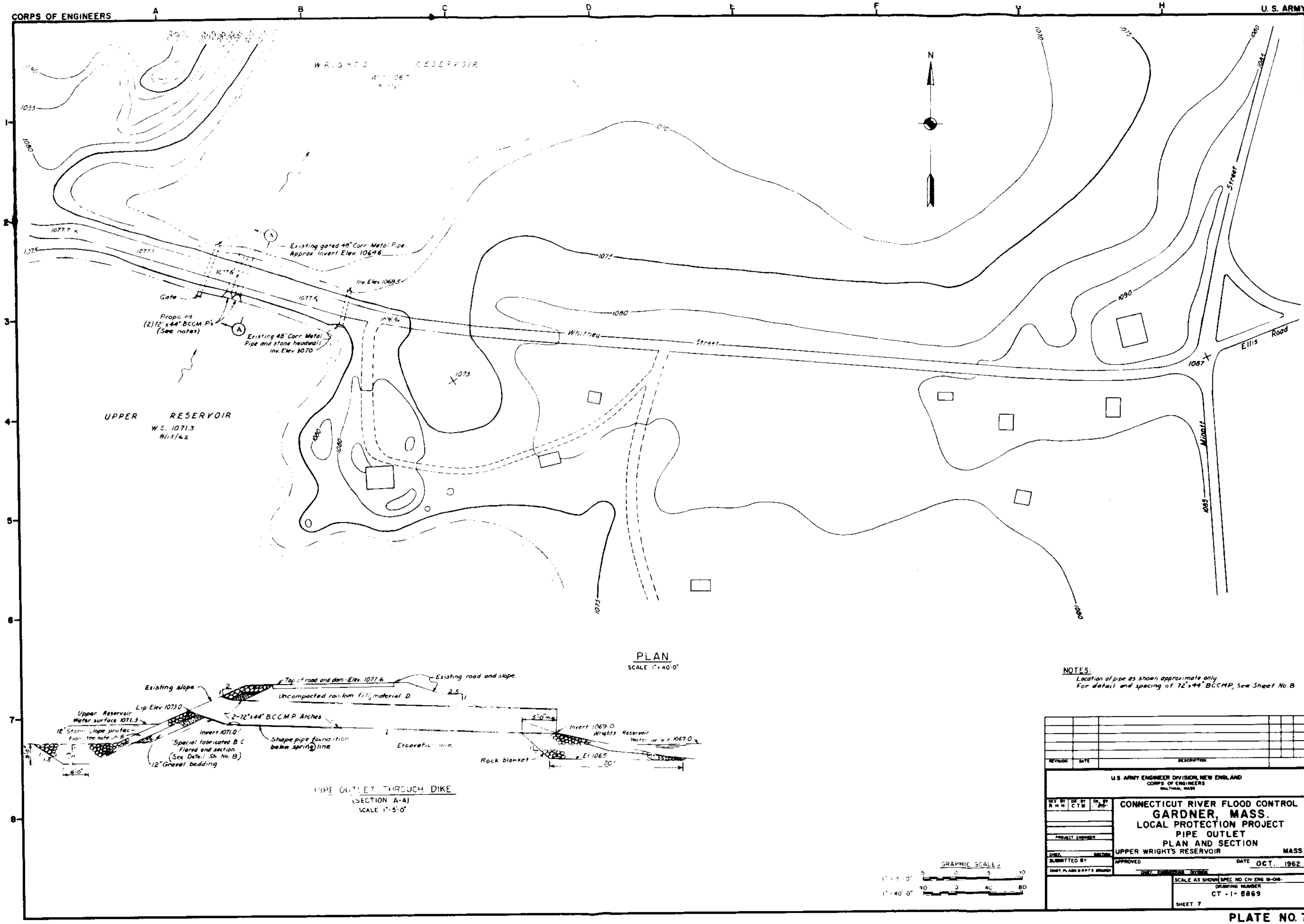


PLATE NO. 5


WRIGHTS RESERVOIR
W.S. 1067
8/15/62
Existing Stone wall and Barbed Wire Fence
New Road Location on top of dike
Top of Dike El. 1076
FD-1
FD-2
FD-3
Mixed Woods
Existing Earth Roadway
PLAN
SCALE: 1"=40'-0"
Landside
Waterside
16'-0"
12'-0"
12" Compacted Gravel
6" Seeded Topsoil
El. 1076
Compacted Impervious Fill
6" Seeded Topsoil
El. 1068±
Original ground
Stripping Line
W.S. El. 1067±
Compacted Gravel Fill above water
Dumped Gravel Fill below water
SECTION A-A
SCALE: 1" = 10'-0"
Top of Dike El. 1076
Original ground - East Toe of Dike
Original ground - ℄ Proposed Dike
Original ground - West Toe of Dike
℄ PROFILE
SCALE: Hor. 1" = 40'-0" Vert. 1" = 4'-0"
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
CONNECTICUT RIVER FLOOD CONTROL
GARDNER, MASSACHUSETTS
LOCAL PROTECTION PROJECT
WRIGHTS RESERVOIR - WEST DIKE
PLAN - PROFILE - SECTION
WRIGHTS RESERVOIR MASSACHUSETTS
OCT. 1962
CT-1-5868



CORPS OF ENGINEERS
U. S. ARMY
WRIGHTS RESERVOIR
UPPER RESERVOIR
W.S. 1071.3
8/15/62
Existing gated 48" Corr. Metal Pipe
Approx. Invert Elev. 1064.6
Inv. Elev. 1069.5
Gate
Propo ed (2) 72"x44" BCCM P's (See notes)
Existing 48" Corr. Metal Pipe and stone headwall Inv. Elev. 1070
Whitney Street
Ellis Road
Minott Street
PLAN
SCALE 1"=40'-0"
NOTES:
Location of pipe as shown approximate only.
For detail and spacing of 72"x44" BCCMP, See Sheet No. 8
Existing slope
Top of road and dam - Elev. 1077.6
Existing road and slope
Uncompacted random fill, material D
Lip Elev 1073.0
Upper Reservoir Water surface 1071.3
2-72"x44" BCCMP Arches
Invert 1071.0
Special fabricated B.C. Flared end section. (See Detail Sh. No. 8)
12" Gravel bedding
Shape pipe foundation below spring line
Invert 1069.0
Wrights Reservoir
Rock blanket
El 1065
PIPE OUTLET THROUGH DIKE
(SECTION A-A)
SCALE 1"=5'-0"
GRAPHIC SCALES
REVISION
DATE
DESCRIPTION
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
DES. BY R H H
DR. BY C T M
CONNECTICUT RIVER FLOOD CONTROL
GARDNER, MASS.
LOCAL PROTECTION PROJECT
PIPE OUTLET
PLAN AND SECTION
UPPER WRIGHTS RESERVOIR
MASS.
PROJECT ENGINEER
SUBMITTED BY
APPROVED
DATE OCT. 1962
SCALE AS SHOWN
DRAWING NUMBER
CT-1-5869
SHEET 7

PLATE NO. 7

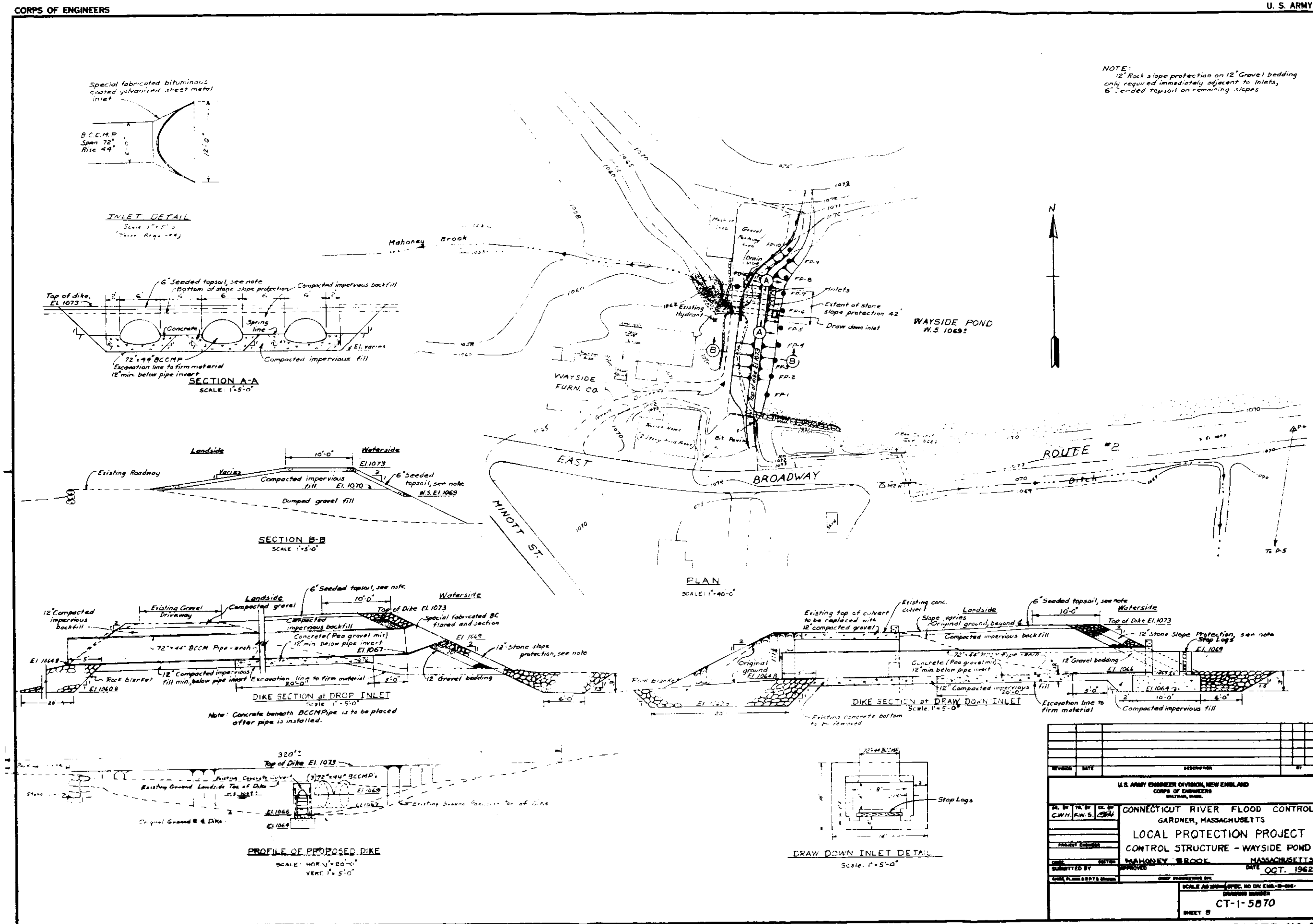

NOTE: 12" Rock slope protection on 12" Gravel bedding only required immediately adjacent to inlets, 6" Seeded topsoil on remaining slopes.
Special fabricated bituminous coated galvanized sheet metal inlet
B.C.C.M.P. Span 72" Rise 44"
INLET DETAIL
SECTION A-A
SCALE: 1"=5'-0"
Top of dike, El. 1073
6" Seeded topsoil, see note
Compacted impervious backfill
Spring line
72"x44" BCCMP
Excavation line to firm material 12" min. below pipe invert
Compacted impervious fill
El. varies
SECTION B-B
SCALE: 1"=5'-0"
Landside
Waterside
Existing Roadway
Compacted impervious fill
Dumped gravel fill
W.S. El. 1069
Mahoney Brook
WAYSIDE POND
W.S. 1069±
Existing Hydrant
Inlets
Extent of stone slope protection 42'
Draw down inlet
WAYSIDE FURN. CO.
EAST BROADWAY
ROUTE #2
MINOTT ST.
PLAN
SCALE: 1"=40'-0"
DIKE SECTION at DROP INLET
Scale: 1"=5'-0"
Note: Concrete beneath BCCMPipe is to be placed after pipe is installed.
DIKE SECTION at DRAW DOWN INLET
Scale: 1"=5'-0"
Stop Logs
PROFILE OF PROPOSED DIKE
SCALE: HOR. 1"=20'-0" VERT. 1"=5'-0"
320'±
Top of Dike El. 1073
(3) 72"x44" BCCMP's
DRAW DOWN INLET DETAIL
Scale: 1"=5'-0"
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.
CONNECTICUT RIVER FLOOD CONTROL
GARDNER, MASSACHUSETTS
LOCAL PROTECTION PROJECT
CONTROL STRUCTURE - WAYSIDE POND
MAHONEY BROOK MASSACHUSETTS
OCT. 1962
CT-1-5870

PLATE NO. 8

CORPS OF ENGINEERS U. S. ARMY

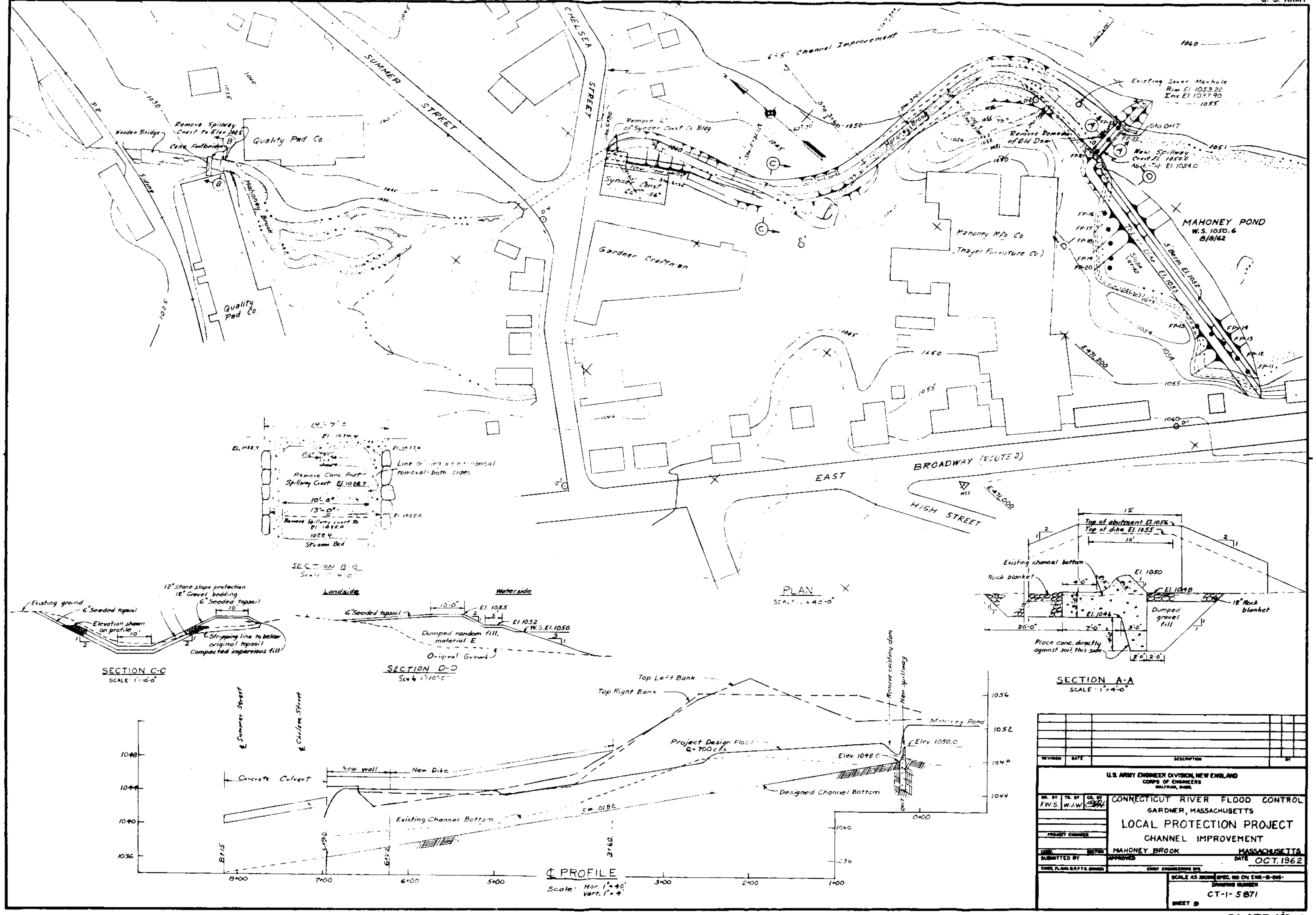
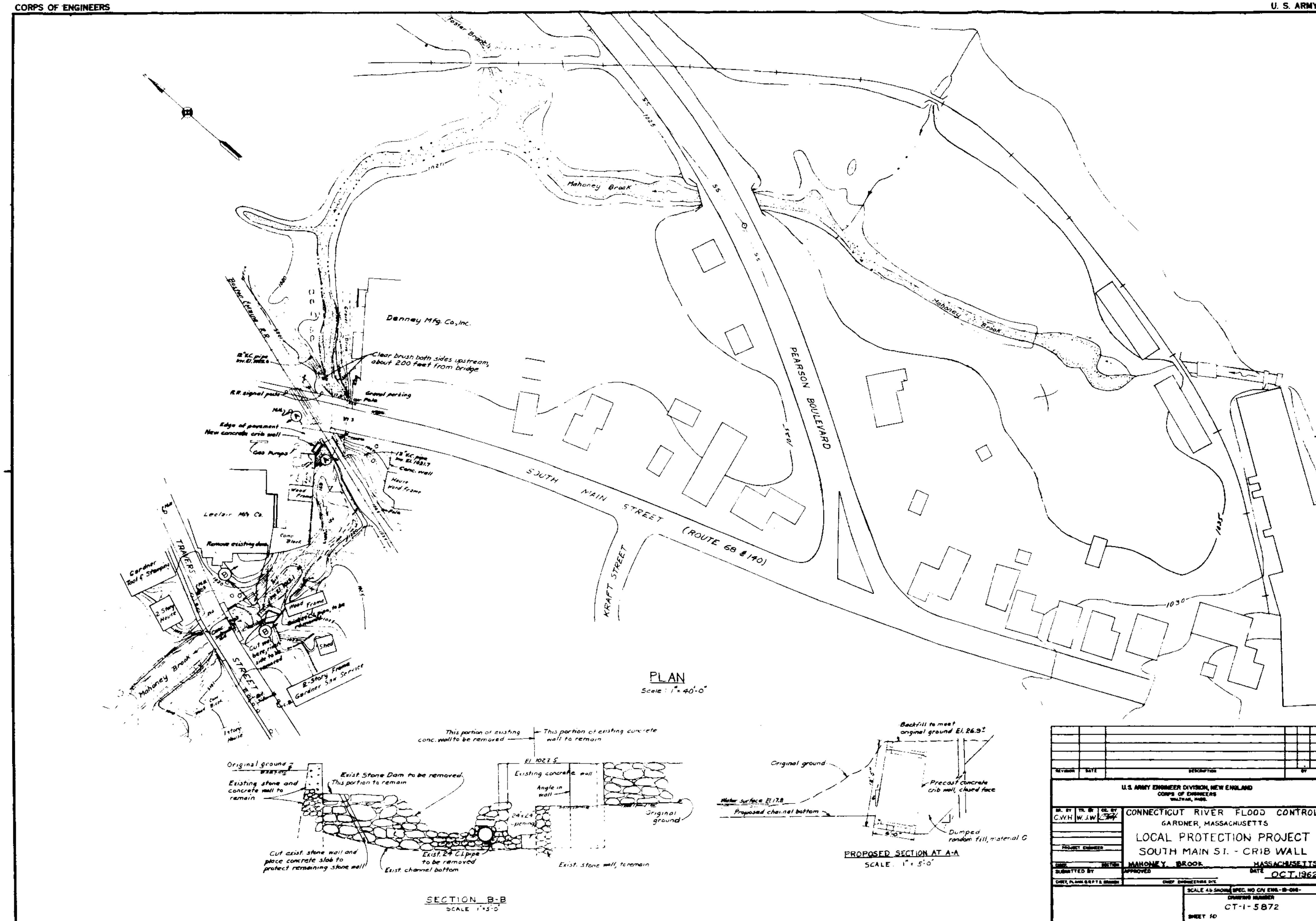
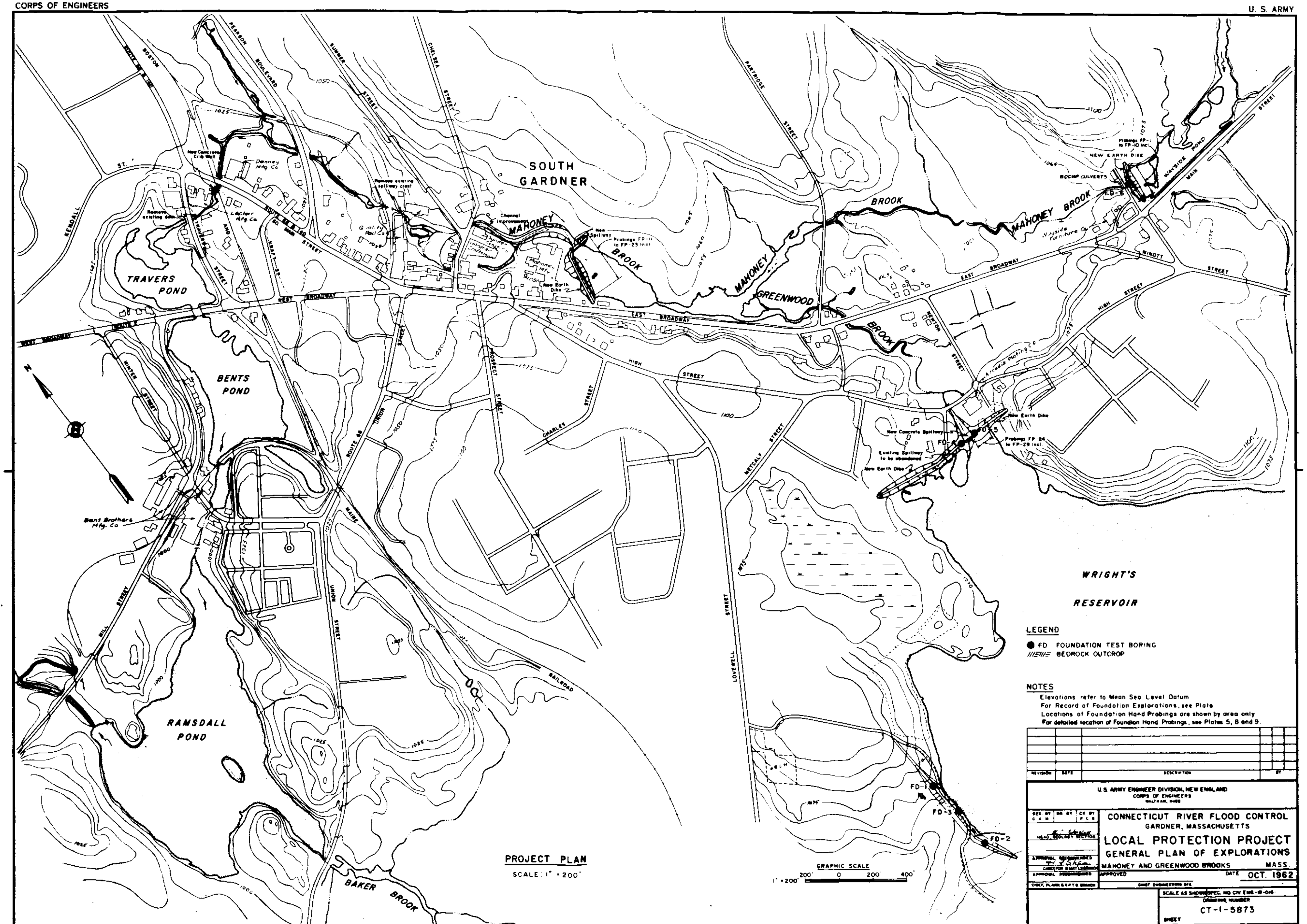
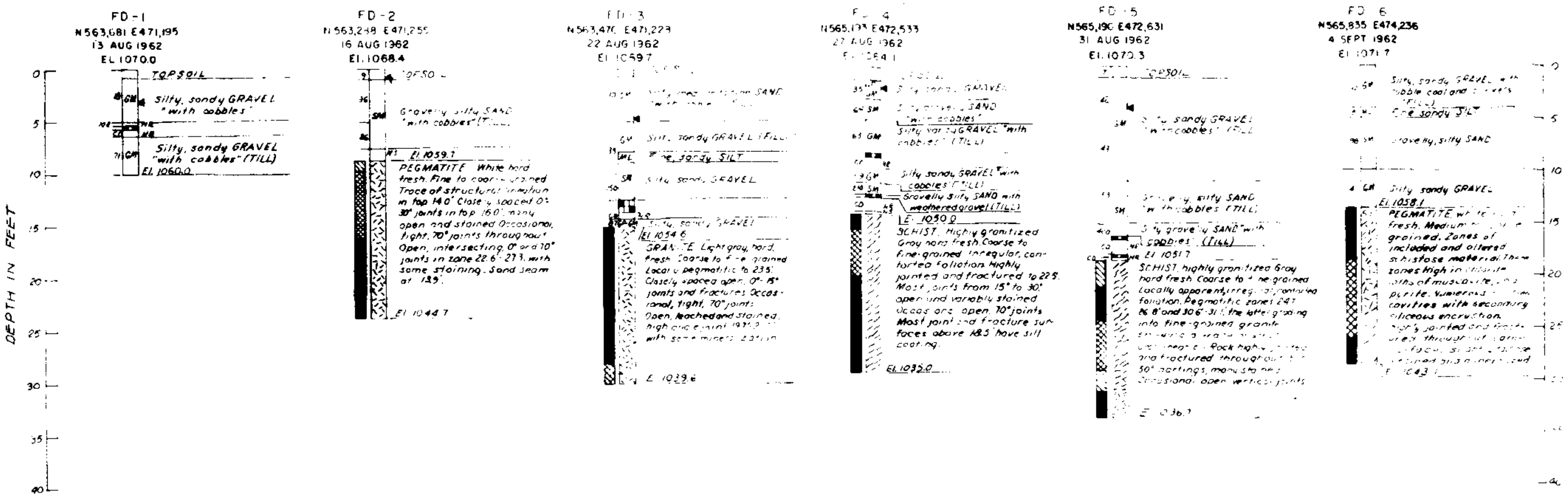


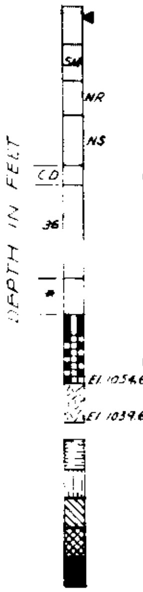
PLATE NO. 9


Foster Brook
Mahoney Brook
Denney Mfg. Co., Inc.
Clear brush both sides upstream, about 200 feet from bridge
Gravel parking
R.R. signal posts
Edge of pavement
New concrete crib wall
Gas Pumps
Conc. wall
Leclair Mfg. Co.
Remove existing dam
Gardner Tool & Stamping
2 Story House
Wood Frame
Shed
2-Story Frame Gardner Saw Service
TRAVERS STREET
SOUTH MAIN STREET (ROUTE 68 & 140)
KRAFT STREET
PEARSON BOULEVARD
PLAN
Scale: 1" = 40'-0"
This portion of existing conc. wall to be removed
This portion of existing concrete wall to remain
Original ground
Existing stone and concrete wall to remain
Exist. Stone Dam to be removed. This portion to remain
Existing concrete wall
Angle in wall
24x24 opening
Original ground
Cut exist. stone wall and place concrete slab to protect remaining stone wall
Exist. 24 C.I. pipe to be removed
Exist. channel bottom
Exist. stone wall, to remain
SECTION B-B
SCALE 1" = 5'-0"
Backfill to meet original ground El. 26.9±
Original ground
Precast concrete crib wall, closed face
Water surface El. 17.8
Proposed channel bottom
Dumped random fill, material G
PROPOSED SECTION AT A-A
SCALE 1" = 5'-0"
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
CONNECTICUT RIVER FLOOD CONTROL
GARDNER, MASSACHUSETTS
LOCAL PROTECTION PROJECT
SOUTH MAIN ST. - CRIB WALL
MAHONEY BROOK
MASSACHUSETTS
OCT. 1962
CT-1-5872


SOUTH
GARDNER
MAHONEY
BROOK
MAHONEY BROOK
GREENWOOD
BROOK
TRAVERS
POND
BENTS
POND
RAMSDALL
POND
BAKER BROOK
WRIGHT'S
RESERVOIR
WAYSIDE POND
WEST BROADWAY
EAST BROADWAY
HIGH STREET
MAIN STREET
MINOTT STREET
UNION STREET
PROSPECT STREET
CHARLES STREET
METCALF STREET
LOVEWELL STREET
WINTER STREET
MILL STREET
PARTRIDGE STREET
SUMMER STREET
CHELSEA STREET
PEARSON BOULEVARD
RAILROAD
Bent Brothers Mfg. Co
Leclair Mfg. Co
Dunney Mfg. Co
Wayside Furniture Co
Arcadia Plating Co
New Earth Dike
New Concrete Spillway
Existing Spillway to be abandoned
New Spillway
Probings FP-11 to FP-23 incl
Probings FP-24 to FP-29 incl
Probings FP-1 to FP-10 incl
NEW EARTH DIKE
FD-1
FD-2
FD-3
FD-5
LEGEND
FD FOUNDATION TEST BORING
BEDROCK OUTCROP
NOTES
Elevations refer to Mean Sea Level Datum
For Record of Foundation Explorations, see Plate
Locations of Foundation Hand Probings are shown by area only
For detailed location of Foundion Hand Probings, see Plates 5, 8 and 9.
REVISION
DATE
DESCRIPTION
BY
U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS
CONNECTICUT RIVER FLOOD CONTROL
GARDNER, MASSACHUSETTS
LOCAL PROTECTION PROJECT
GENERAL PLAN OF EXPLORATIONS
MAHONEY AND GREENWOOD BROOKS MASS.
HEAD, GEOLOGY SECTION
APPROVED
DATE OCT. 1962
CHIEF, ENGINEERING DIV.
DRAWING NUMBER
CT-1-5873
SHEET
PROJECT PLAN
SCALE: 1" = 200'
GRAPHIC SCALE
1" = 200'
200' 0 200' 400'



LEGEND FOR GRAPHIC LOGS



FD-3	Foundation Test Boring
N563,470 E471,228	Location by coordinates
22 AUG 1962	Date exploration completed
El 1069.7	Elevation of ground surface during time of exploration
	Subsurface water level in boring at time of exploration
SM	Group letter symbol according to Unified Soil Classification System
NR	No Recovery or unsatisfactory soil samples recovered.
NS	Not Sampled. Hole advanced by Core-drilling, blasting and/or wash boring due to operational difficulty
CD	Core-drilled in overburden
36	Blows per foot of penetration considered most representative for each sample drive using a 300 or 350 pound hammer with a free fall of about 18 inches on a 2½" O.D. or 3" O.D. and 2" I.D. or 2⅜" O.D. size sample spoon equipped with a beveled and sharpened drive shoe.
*	Blow counts not recorded or not considered representative
	Cobble or boulder (Core drilled)
	Cobbles or boulders continuous or nested (Core-drilled and/or blasted and chopped)
El 1054.6	Elevation of bedrock surface
	Rock symbol
El 1039.6	Elevation bottom of exploration
	Rock core recovery 0 - 25%
	Rock core recovery 25 - 50%
	Rock core recovery 50 - 75%
	Rock core recovery 75 - 90%
	Rock core recovery 90 - 100%

Water levels recorded during subsurface explorations seldom correspond with the natural level of free ground water except in extensive and thick deposits of sands and gravels which are sufficiently pervious to permit rapid stabilization of water levels in the exploratory hole. Absence of subsurface water level in the graphic log of any exploration is not necessarily to be construed that ground water will not be encountered in excavation at that location.

While the borings are representative of subsurface conditions at their respective locations and for their respective vertical reaches, local minor variations characteristic of the overburden and rocks of this region are anticipated and, if encountered, such variations will not be considered as differing "Materially" from represented conditions within the purview of Article 4 of the contract.

NOTES

Elevations refer to Mean Sea Level Datum

For general location of Foundation Explorations, see Plate 11

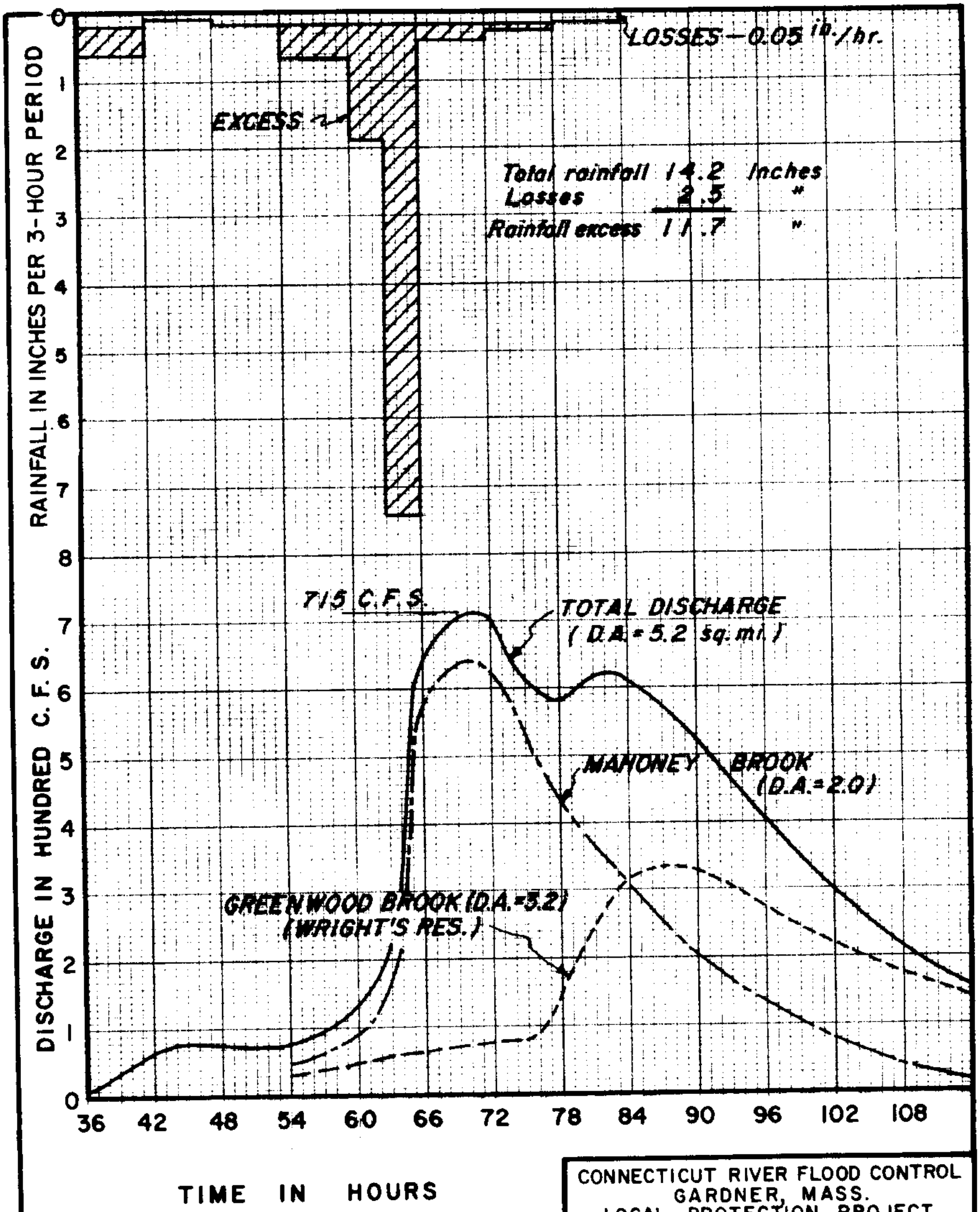
For detailed location of Foundation Explorations, see Plates 5, 8 and 9

PROBING NUMBER	GROUND ELEV.	PENETRATION WEIGHT OF 1 MAN (a)	WEIGHT OF 1 MAN (b)	WEIGHT OF 2 MEN (a)	WEIGHT OF 2 MEN (b)	DEPTHS AND BLOWS / FT. REQ'D (BLE HAMMER) (a)	(b)	(c)	REFUSAL
FP-1	[illegible]	[illegible]	2.5	—	—	0.5	4.0	[illegible]	—
FP-2	1067.[illegible]	[illegible]	2.4	[illegible]	1.1	1.1	[illegible]	[illegible]	—
FP-3	1065.[illegible]	[illegible]	[illegible]	—	—	[illegible]	4.[illegible]	[illegible]	—
FP-4	[illegible]7.4	[illegible]	[illegible]	2.2	2.3	2.3	4.[illegible]	[illegible]	
FP-5	1068.5	[illegible]	[illegible]	[illegible]	[illegible]	2.6	3.[illegible]	[illegible]	
FP-6	[illegible]	[illegible]	[illegible]		—	[illegible]4	5.[illegible]	[illegible]	
FP-7	1066.[illegible]	[illegible]	[illegible]	2.8	[illegible]	[illegible]	3.[illegible]	[illegible]	
FP-8	[illegible]	[illegible]	[illegible]		—	[illegible]	3	[illegible]	
FP-9	[illegible]	[illegible]	8	[illegible]	[illegible]	2.[illegible]	4.[illegible]	[illegible]	
FP-10	1067.5	[illegible]	[illegible]6	—	—	[illegible]6	4.[illegible]	[illegible]	—
FP-11	[illegible]	[illegible]	[illegible]	—	—	2.[illegible]	4	[illegible]	—
FP-12	[illegible]	[illegible]	[illegible]	—	—	1.3	3.[illegible]	[illegible]	
FP-13	[illegible]4	[illegible]	4	4	6	1.6	4.[illegible]	[illegible]	
FP-14	[illegible]	[illegible]	[illegible]	—	—	2.0	4.0	[illegible]	—
FP-15	[illegible]7.8	[illegible]	3.7	—	—	3.7	5.0	[illegible]	
FP-16	[illegible]7.2	[illegible]	[illegible]		—	[illegible]	4.[illegible]	[illegible]	
FP-17	104[illegible].2	[illegible]	[illegible]		—	[illegible]2	[illegible]	[illegible]	—
FP-18	1047.[illegible]			[illegible]	8	[illegible]	3.[illegible]	[illegible]	
FP-19	[illegible]		[illegible]	—	—	[illegible]	4.[illegible]	[illegible]	
FP-20	1046.8	[illegible]	[illegible]	—	—	[illegible]4	4.[illegible]	[illegible]	—
FP-21	105[illegible].6			—	—	[illegible]	3.[illegible]	[illegible]	—
FP-22	[illegible]8.9	[illegible]	[illegible]	—	—	[illegible]	[illegible]	[illegible]	REFUSAL
FP-23	[illegible]2.8	—	—	—	—	[illegible]	[illegible]	[illegible]	—
FP-24	[illegible]	[illegible]	2	—	—	[illegible]2	[illegible]	[illegible]	REFUSAL
FP-25	1061.5	[illegible]	5	—	—	[illegible]5	3.[illegible]	[illegible]	—
FP-26	1060.8	[illegible]	[illegible]	—	—	[illegible]	[illegible]	[illegible]	
FP-27	1060.2	[illegible]	3.[illegible]	—	—	3.[illegible]	4.0	[illegible]	—
FP-28	1060.[illegible]	[illegible]	[illegible]5	—	—	[illegible]	4.[illegible]	[illegible]	—
FP-29	1060.[illegible]	[illegible]	[illegible]6	—	—	[illegible]	3.[illegible]	[illegible]	—

REVISION | DATE | DESCRIPTION | BY

U.S. ARMY ENGINEER DIVISION, NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

CONNECTICUT RIVER FLOOD CONTROL
GARDNER, MASSACHUSETTS
LOCAL PROTECTION PROJECT
RECORD OF FOUNDATION EXPLORATIONS
MAHONEY AND GREENWOOD BROOKS — MASS.
OCT. 1962
SPEC. NO. CIV. [illegible]
DRAWING NUMBER
CT-1-5875

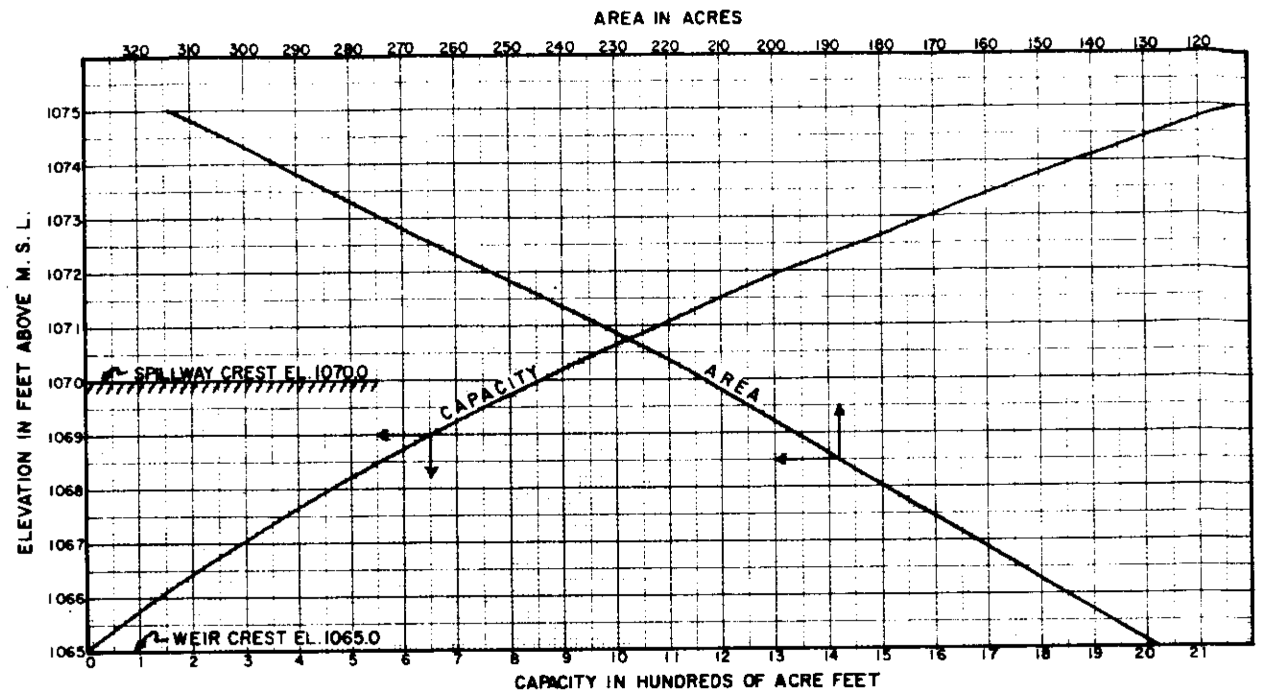


CONNECTICUT RIVER FLOOD CONTROL
GARDNER, MASS.
LOCAL PROTECTION PROJECT

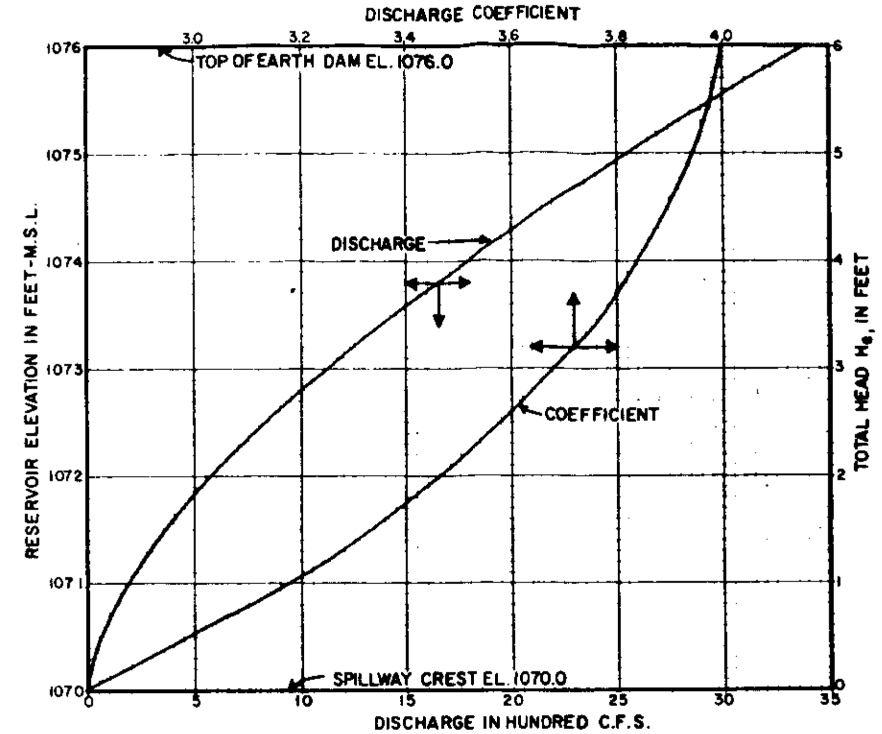
STANDARD PROJECT FLOOD

MAHONEY BROOK AT MOUTH OF GREENWOOD BROOK

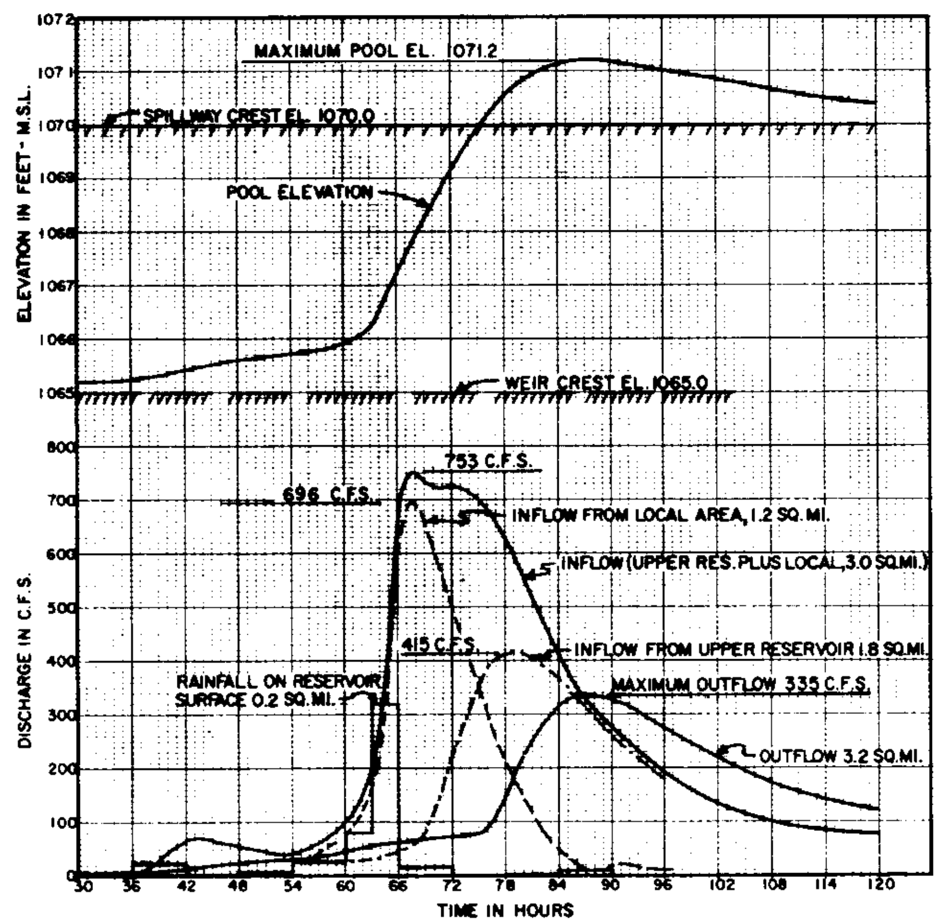
U.S. Army Engineer Division, New England
Corps of Engineers Waltham, Mass.
OCTOBER 1962



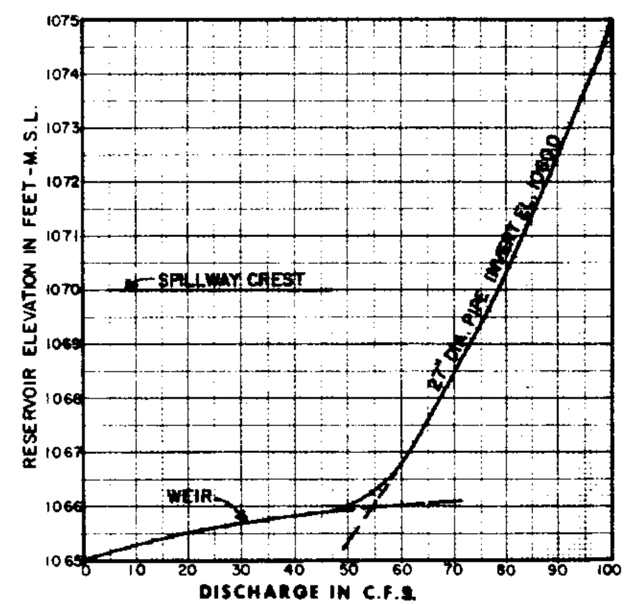
AREA & CAPACITY CURVES



SPILLWAY RATING CURVE



STANDARD PROJECT FLOOD



WEIR & OUTLET RATING CURVE

CONNECTICUT RIVER FLOOD CONTROL
GARDNER, MASS.
LOCAL PROTECTION PROJECT
WRIGHTS RESERVOIR
HYDROLOGY & HYDRAULICS
NEW ENGLAND DIVISION WALTHAM, MASS.
OCTOBER 1962

APPENDIX A

LETTERS OF CONCURRENCE AND COMMENT

Exhibit No.	Agency	Letter Dated
1	City of Gardner, Massachusetts Approval of City Council	November 25, 1960
2	U. S. Department of Agriculture Soil Conservation Service	August 22, 1961
3	U. S. Department of Agriculture Soil Conservation Service	October 19, 1961
4	U. S. Department of Agriculture Soil Conservation Service	May 7, 1962
5	U. S. Department of the Interior Fish and Wildlife Service	November 24, 1961
6	U. S. Department of the Interior Fish and Wildlife Service	July 10, 1962
7	U. S. Department of the Interior Fish and Wildlife Service	August 24, 1962

CITY OF GARDNER
MASSACHUSETTS
OFFICE OF
CITY COUNCIL

CHAIR CITY OF THE WORLD

November 25, 1960.

PRESIDENT
EDWARD W. TAMULEN

COUNCILLORS AT LARGE
ULRIC O. FREDETTE
ARNOLD E. LUCAS
ROSAIRE J. ST. JEAN
ATTY. LOUIS SCERRA
JOHN P. SULLIVAN
DR. TOIVO TIIHONEN

WARD COUNCILLORS
WARD 1
CLARKE K. STEDMAN
WARD 2
EDMOND GAUTREAU
WARD 3
CLARENCE J. CORMIER
WARD 4
GUY R. SHARRON
WARD 5
EDWARD W. TAMULEN

Brig. Gen. Alden K. Sibley
U. S. Army Division Engineer
424 Trapelo Road
Waltham, Mass.

Dear Sir:

Enclosed is a copy of the Council vote expressing its willingness to go along with measures for flood prevention at Pew and Nigger Brooks in Gardner.

The action was taken at the regular meeting of the City Council on November 21, 1960 and approved by Mayor Landry.

Very truly yours,

Genia J. Pacocha

(Miss) Genia J. Pacocha
City Clerk.

EXHIBIT NO. 1

APPROVAL OF PEW AND NIGGER BROOKS

FLOOD CONTROL PROJECT

VOTED: That the City Council go on record as favoring and willing to go along with a Federal Flood Control Project at Pew and Nigger Brooks in Gardner.

Vote

PEW AND NIGGER BROOKS
FLOOD CONTROL PROJECT
GARDNER, Mass.

In City Council
Nov. 21, 19 60

Vote Passed Nov. 21, 19 60

Clerk

Presented to Mayor for approval
Nov. 23, 19 60

Approved NOV 25 1960 19

Mayor

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

29 Cottage Street, Amherst, Massachusetts

August 22, 1961

Mr. John Wm. Leslie
Chief, Engineering Division
U. S. Army Engineer Division
424 Trapelo Road
Waltham 54, Massachusetts

Dear Mr. Leslie:

We have reviewed the two plans for a proposed local flood protection project at Gardner, Massachusetts outlined in your letter of July 26, 1961. Because there was no specific data submitted, our comments necessarily will be very general.

An examination of the U.S.G.S. Topographic Quadrangles indicates that a system of small detention reservoirs should also be considered. There appear to be several reservoir sites available that could effectively control a large part of the drainage area above the damage center. We feel that a plan of stream channel improvement or diversions, and a plan of a system of reservoirs should both be studied to determine their relative merits in meeting the objectives of the local people.

I should like to suggest that personnel from both agencies get together to consider the proposals in greater detail. We would then be in a position to make comments and determine what assistance we could offer. We would be able to schedule a joint review sometime after the middle of September.

Sincerely yours,

Benj. Isgur
Benjamin Isgur,
State Conservationist.

Growth Through Agricultural Progress

EXHIBIT NO. 2

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
29 Cottage Street, Amherst, Massachusetts

October 19, 1961

Division Engineer
New England Corps of Engineers
424 Trapelo Road
Waltham 54, Massachusetts

Attention: Chief, Engineering Division

Dear Sir:

Mr. Mills and Mr. McLaren of my staff reviewed your proposals for Pew Brook at Gardner, Massachusetts, on September 25, 1961. From the material available it appears that a feasible project could be developed under the authorities available to either of our agencies. Unless we receive an application from the local interests we will not be officially concerned with this project.

As was mentioned in my previous letter to you, it is felt that a system of reservoirs should be given serious consideration. The solution of the flood control problem by the use of reservoirs would afford flood protection and would afford an opportunity for the development of multiple-purpose structures in the watershed. We shall be glad to cooperate in any way we can.

Sincerely yours,

Benj. Isgur

Benjamin Isgur
State Conservationist.

EXHIBIT NO. 3

UNITED STATES DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

29 Cottage Street, Amherst, Massachusetts

May 7, 1962

Mr. John Wm. Leslie
Chief, Engineering Division
U. S. Army Corps of Engineers
424 Trapelo Road
Waltham 54, Massachusetts

Dear Mr. Leslie:

Thank you for your letter of April 19, 1962, regarding the proposed flood control project on Pew Brook in Gardner, Massachusetts.

I have no additional comments to those made in my letters of August 22 and October 19, 1961.

Sincerely yours,

/s/ Benj. Isgur

Benjamin Isgur,
State Conservationist.

Growth Through Agricultural Progress

EXHIBIT NO. 4

UNITED STATES
DEPARTMENT OF THE INTERIOR
FISH AND WILDLIFE SERVICE
BUREAU OF SPORT FISHERIES AND WILDLIFE
59 TEMPLE PLACE
BOSTON, MASSACHUSETTS

ADDRESS ONLY THE
REGIONAL DIRECTOR

NORTHEAST REGION
(REGION 5)
MAINE
NEW HAMPSHIRE
NEW YORK
VERMONT
PENNSYLVANIA
MASSACHUSETTS
NEW JERSEY
RHODE ISLAND
DELAWARE
CONNECTICUT
WEST VIRGINIA

November 24, 1961

Division Engineer
New England Division
U. S. Army Corps of Engineers
424 Trapelo Road
Waltham 54, Massachusetts

Dear Sir:

Reference is made to your letter of July 29, 1961 transmitting to this office the plans you are considering for the local flood protection project at Gardner, Massachusetts, and in which you solicit comments from this office on the effect of the project on the fish and wildlife resources. This letter constitutes our conservation and development report on the project and has the concurrence of the Massachusetts Division of Fisheries and Game.

It is our understanding that 2 plans of improvement are under consideration. One plan would provide for the diversion of Pew Brook to the south, thereby bypassing the industrial area of South Gardner. This plan would require a control structure at Route 2 that would permit flood waters from Pew Brook to enter a channel and flow into Wright's Reservoir, and a sluiceway on the western shore of the reservoir that would convey overflow westward into Baker Brook thence into Ramsdell Pond and the Otter River. The plan would also require reconstruction of culverts at 4 roads and one railroad crossing as well as the reconstruction and modification of 3 privately owned dams. An alternate plan would require the widening of the existing Pew, Nigger, and Foster Brook channels to Mill Street.

Fishery values involved are low. However, the area within the influence of the project includes a considerable amount of waterfowl habitat which is dependent upon the impounded shoal waters in Pew Brook, Wright's Reservoir, and Ramsdell Pond being maintained at their present, normal levels. Your plan as presently proposed does not indicate that you are considering maintaining these impoundments at a lower level, but we wish to call to your attention that lowering levels in these areas would result in significant wildlife losses.

Wright's Reservoir is a Massachusetts Great Pond to which there is no public access. The Massachusetts Division of Fisheries and Game has an interest in managing the reservoir fishery but until public access is provided no fishery management program can be initiated. It is felt that minimum public access

EXHIBIT NO 5

could be provided as an incidental project benefit with no additional cost by merely keeping in mind such access as the finishing work is being accomplished either at the proposed sluiceway on the western shore of the reservoir or at the reservoir dam which would be reconstructed. This reservoir would attract only the local residents and it is not expected that any heavy use problem would arise from this minimum access. Should the local people desire more elaborate access as the project plan develops, such access would be negotiated for at local level. The Massachusetts Division of Fisheries and Game would assist in such negotation. The access feature would, of course, be contingent upon local approval and acceptance of this diversion plan for flood protection.

Your plans of improvement as presently envisioned would have no detrimental effect on the fish and wildlife resources, and no further report on this project from this agency is considered necessary. Should you change your plans, however, to include maintaining the impoundments at a lower level, we would appreciate being notified in order that a new fish and wildlife report may be prepared.

Thank you for the opportunity to report on this project.

Sincerely yours,

R P Boone

R. P. Boone
Acting Regional Director

ADDRESS ONLY THE
REGIONAL DIRECTOR

UNITED STATES
DEPARTMENT OF THE INTERIOR
FISH AND WILDLIFE SERVICE
BUREAU OF SPORT FISHERIES AND WILDLIFE
59 TEMPLE PLACE
BOSTON, MASSACHUSETTS

NORTHEAST REGION
(REGION 5)
MAINE
NEW HAMPSHIRE
NEW YORK
VERMONT
PENNSYLVANIA
MASSACHUSETTS
NEW JERSEY
RHODE ISLAND
DELAWARE
CONNECTICUT
WEST VIRGINIA

July 10, 1962

Division Engineer
U. S. Army Corps of Engineers
New England Division
424 Trapelo Road
Waltham 54, Massachusetts

Dear Sir:

Your letter of April 19, 1962 informed us that your original plans for local protection at Gardner, Massachusetts, had been revised. The fish and wildlife conservation and development report which we sent to you on November 24, 1961, was based on your original plans. This letter is a supplement to the earlier report and considers the fish and wildlife aspects of your new plans. It was prepared under the authority of the Fish and Wildlife Coordination Act (48 Stat. 401, as amended; 16 U.S.C. 661 et seq.), in cooperation with the Massachusetts Division of Fisheries and Game. It has the concurrence of that agency as indicated in its letter of July 9, 1962.

It is our understanding that your new plan of improvement at Gardner consists of a modification to Wrights Dam and Reservoir with downstream channel improvements on Nigger Brook and Pew Brook. This plan would provide for a new dam, spillway, and outlet at the north end of Wrights Reservoir, a flanking dike along the west shore of the reservoir, strengthening of the existing outlet structure at Pew Brook, widening and deepening of Pew Brook at critical locations, and the removal of several channel obstructions.

The impounded waters of Pew Brook and Ramsdell Pond would be maintained at their present normal levels. However, the impounded water level on Wrights Reservoir would be altered slightly and possibly become somewhat more stabilized by the newly proposed plan of improvement. The channel work envisioned in your new plans would have no detrimental effect on the fish and wildlife resources.

In our conservation and development report, we stated a need for public access to Wrights Reservoir and suggested that minimum access could be provided as an incidental project benefit. In his letter dated June 14, 1962, Mr. Leslie of your staff advised us that under provisions of Public

EXHIBIT NO. 6

Law 685, lands and rights-of-way required for construction, maintenance, and operation are provided by the City and that ownership of land and control of completed works would rest with the City. For these reasons, the Federal Government would have no control of access into the area. Mr. Leslie also suggested that Game interests discuss the access feature with City officials.

In checking into the matter of water rights in Wrights Reservoir, it has been found that these rights are controlled by Arcadia Manufacturing Company, Inc., of Gardner. Initial contact with this company indicates that it discourages fisherman-use of the reservoir.

If water rights are to remain vested in Arcadia Manufacturing Company, Inc., following project completion, favorable consideration of access facilities appears unlikely, although we would hesitate to say it could not be brought about. If, however, the City is going to acquire the water rights as a result of the local protection works, then a meeting to discuss provisions for fisherman access would be desirable.

We would appreciate your advising us as to the future status of the water rights at Wrights Pond. If the City of Gardner is going to acquire the rights, we will then arrange to meet with your staff, City officials, and Massachusetts Division of Fisheries and Game personnel to discuss access facilities.

Sincerely yours,

John S. Gottschalk
Regional Director

UNITED STATES
DEPARTMENT OF THE INTERIOR
FISH AND WILDLIFE SERVICE
BUREAU OF SPORT FISHERIES AND WILDLIFE
59 TEMPLE PLACE
BOSTON, MASSACHUSETTS

ADDRESS ONLY THE
REGIONAL DIRECTOR

NORTHEAST REGION
(REGION 5)
MAINE
NEW HAMPSHIRE
NEW YORK
VERMONT
PENNSYLVANIA
MASSACHUSETTS
NEW JERSEY
RHODE ISLAND
DELAWARE
CONNECTICUT
WEST VIRGINIA

August 24, 1962

Division Engineer
U.S. Army Engineer Division, N.E.
Corps of Engineers
424 Trapelo Road
Waltham 54, Mass.

Dear Sir:

Thank you for your letter of August 22, 1962 concerning your local flood protection project at Gardner, Mass. We appreciate the efforts you have made in regard to the subject of public access.

Sincerely yours,

Ennio Abbiati

Ennio Abbiati, Acting Chief
Division of Technical Services

EXHIBIT NO. 7

APPENDIX B

FLOOD LOSSES AND BENEFITS

PLATES

APPENDIX B

FLOOD LOSSES AND BENEFITS

1. DAMAGE SURVEYS

A detailed damage survey was made in early 1961 to determine the extent of damage that would be sustained in South Gardner under current conditions in a recurrence of 1938 flood stages. In addition to obtaining losses for the 1938 flood crest, sufficient data was gathered to derive losses for stages 3 feet higher, the stage where damage begins, and intermediate stages where marked increases in damage occurs. The damage survey consisted essentially of door-to-door interviews and inspections of the residential, commercial, industrial and other properties affected by flooding. The recorded information included the extent of the areas flooded, description of properties, nature and amount of damages, depth of flooding, high water references and relationships to prior flood stages. Generally, damage estimates were furnished by property owners or tenants. Investigators used their own judgement in modifying these estimates and also made estimates where owner or tenant estimates were not available. Sampling methods were used where properties of the same general type were subject to the same depth of flooding.

2. LOSS CLASSIFICATION

Flood loss information was recorded by type of loss and by location. Loss types used were industrial, urban (commercial, residential, public), highway, railroad and utility. Primary losses were evaluated including (1) physical losses, such as damages to structures, machinery and stock, and the cost of cleanup and repairs, and (2) non-physical losses, such as unrecoverable loss of business and wages, cost of temporary facilities and increased cost of operation. The primary loss resulting from physical damages and a large part of the related non-physical loss were determined by direct inspection of property and evaluation of losses by the property owner and field investigators. Where non-physical portions of primary losses could not be determined from available data, estimates were based upon relationships between physical and non-physical losses for similar properties in the area.

3. FLOOD DAMAGES

According to information gathered from state and municipal officials, and interviews with plant engineers, merchants, and local inhabitants, properties located within the flood plain of Mahoney Brook and Foster Branch in South Gardner have been subjected to flooding intermittently for the past 50 years. The most serious flood recorded occurred in September 1938 when over 14 inches of rain fell in a 5-day period. Flooding began when the earth dam at the north end of Wrights Reservoir was overtopped, pouring water into Greenwood and Mahoney Brooks. A building north of the dam, occupied by the old Arcadia Ballroom, was flooded to depths 3 feet over the first floor. In front of the building, a portion of High Street was scoured out. Downstream, river stages some 3 feet above river banks roared through the basement floor of the old Mahoney Plant, upsetting raw and finished goods and overtopping expensive machinery. A warehouse to the rear of the plant was undermined and eventually torn down. At Chelsea Street, flood waters ponded behind a clogged culvert, then broke through, gouging out a part of the roadway and undermining a 5-story building. Directly below this failure, a 3-story building lost 40 feet of its foundation. It was in this vicinity that several thousand chairs washed away to jam at other narrow restrictions downstream.

At South Main Street, beyond the confluence of Mahoney Brook and Foster Branch, floating debris fastened itself at the railroad bridge, damming the water and causing flooding of homes, businesses, and the production floor of the Denny Manufacturing Company plant. At Bents Pond, water overtopped Winter Street and flooded around the buildings of the Bents Brothers Manufacturing Company. Several thousand board feet of lumber were washed into Ramsdall Reservoir. Some spilled over the dam and jammed at the Mill Street culvert, forcing floodwaters into the basement floor of the Ramsdall Mill. Eventually, Mill Street gave way to the force of the ponded water. In addition to the physical losses in the plant areas, the flood forced the shutdown of production and business operations for several weeks. It is estimated that total damages caused by the 1938 flood in South Gardner amounted to $350,000.

4. RECURRING AND PREVENTABLE LOSSES

Stage damage curves, referenced to the peak elevations experienced during the September 1938 flood in the various reaches, were developed to reflect the magnitude of recurring losses of stages above and below the reference flood. The recurring losses used in development of the stage damage relationship reflect economic and physical changes since the 1938 flood. A recurrence of 1938 flood stages would cause damages of $1,050,000 in South Gardner under current conditions. Construction of the proposed local protection project would eliminate $896,000 of these losses.

5. AVERAGE ANNUAL LOSSES

Stage loss data was summarized by reaches that have uniform hydraulic characteristics throughout. Recurring losses in each of the 5 reaches were converted to average annual losses by correlation of stage damage and stage frequency curves to produce damage frequency relationships in accordance with standard Corps of Engineers practice. The average annual loss in the reaches of Mahoney Brook downstream of Wrights Reservoir to Mill Street is estimated at $24,400. Plates B-1 and B-2 show procedures used in converting recurring stage damage data to curves of damage frequency for a typical reach.

6. TRENDS OF DEVELOPMENT

Gardner, Massachusetts has been a static community economically for the past 3 decades. While the State of Massachusetts and Worcester County have experienced growth comparable to that which has taken place nationally, Gardner has had difficulty holding its own. Analysis of population, manufacturing, and retail trade statistics and projection by national and regional agencies leads to the conclusion that additional growth in the area will be slow in coming. Inspection of the area affected by flooding and to be benefited by contemplated protective works reveals an already developed urban area. Industries, commercial establishments, and residences occupy practically all of the flood area. There has been little change in the area in a 30-year period. Community efforts to attract new investment and enterprise are devoted to sites outside the flood area. The relationship of the Gardner population, manufacturing and retail trade statistics are shown on Table I. Trends in Gardner considered in light of trends in other areas and availability of other more desirable lands for development, leads to the conclusion that growth in the project area will be negligible during the project life. Further, provision of flood protection will reduce the recurrence of flood damages without enhancing the value of any vacant lands in the project area.

ESTIMATES OF BENEFITS

7. FLOOD PREVENTION BENEFITS

Flood damage prevention benefits for the South Gardner project were derived by determining the difference in annual losses under pre-project conditions and those existing after project completion. The South Gardner local protection project would reduce the average annual losses in the damage reaches from $24,400 to $9,400 resulting in average annual benefits of $15,000. Benefits were not computed for floods greater than the design flood which has a probability of occurrence of once every 200 years. Derivation of benefits for a typical reach on Mahoney Brook

TABLE I

GARDNER, MASSACHUSETTS

	1930	1940	1950	1960	1965	1970	
Gross National Product (millions of dollars)	91,105	100,618	284,599	504,400	630,000#	790,000#	(#National Planning Association Projections)
National Population (millions)	122.8	131.7	150.7	179.3 169.4*	196.0# 176.3*	214.0# 183.2*	(*NENYIAC Project)
Massachusetts Population (thousands)	4249.6	4316.7	4690.5	5148.6			
Gardner, Massachusetts Population (thousands)	19,399	20,206	19,581				

VALUE ADDED BY MANUFACTURE	1939	1947	1954	1958
Massachusetts thousands of (Actual dollars)	$1,181,465	$3,370,094	$4,356,462	$5,188,706
Massachusetts thousands of (1939 dollars)	1,181,465	1,685,000	1,815,000	1,922,000
Gardner, Mass. thousands of (Actual dollars)	13,039	36,712	26,946	27,980
Gardner, Mass. thousands of (1939 dollars)	13,039	18,355	11,228	10,363

RETAIL SALES	1939	1948	1954	1958
Massachusetts thousands of (Actual dollars)	$1,737,680	$4,302,147	$5,539,100	$6,241,867
Massachusetts thousands of (1939 dollars)	1,737,680	2,151,000	2,308,000	2,312,000
Gardner, Mass. thousands of (Actual dollars)	8,235	20,855	22,344	23,699
Gardner, Mass. thousands of (1939 dollars)	8,235	10,428	9,310	8,777

are shown on plates B-1 and B-2. In addition to preventing tangible losses, the local protection project would produce intangible benefits through reductions of hazards to life and health connected with serious flooding. It would also contribute to the local economy through the prevention of production and business interuptions.

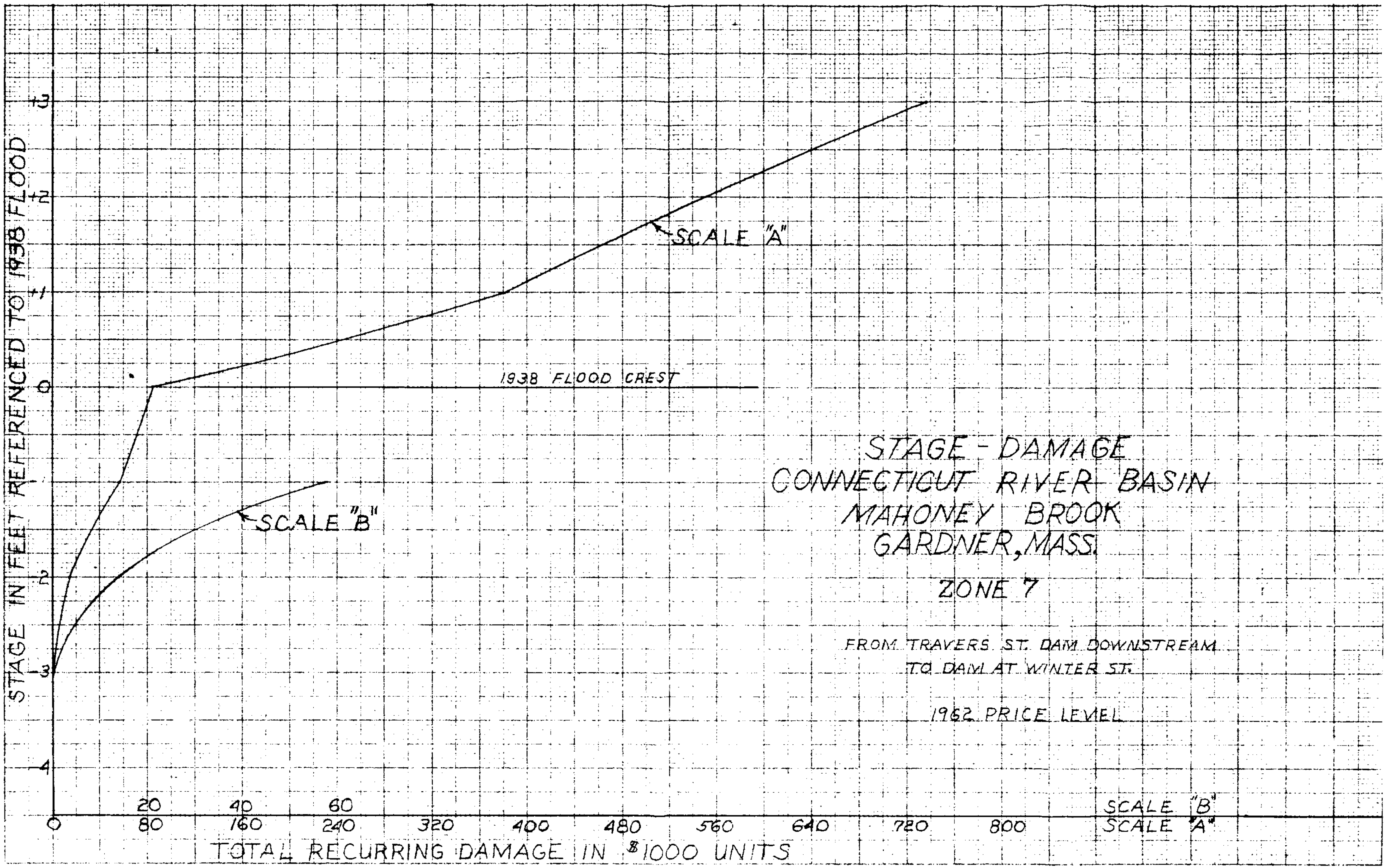
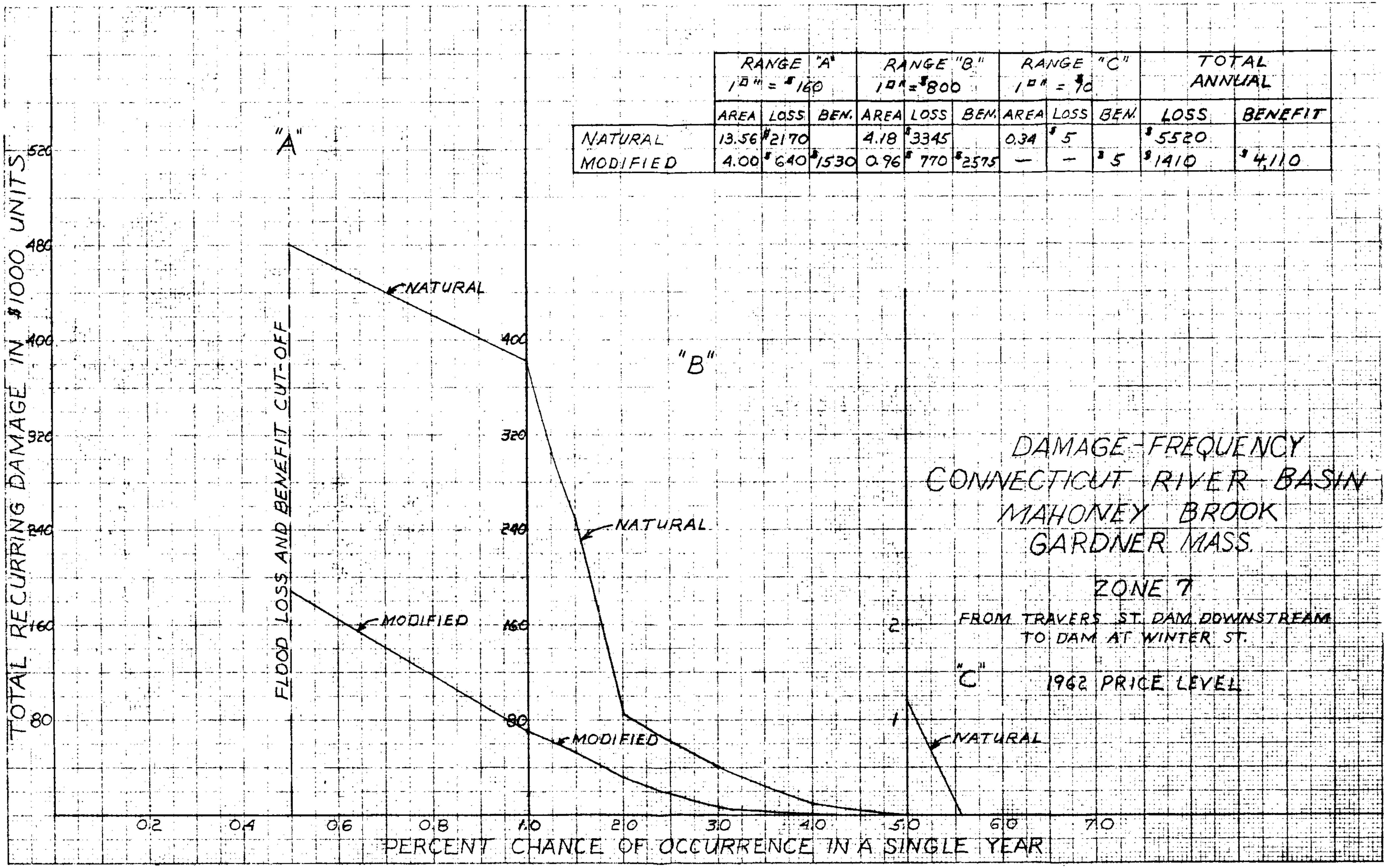

STAGE - DAMAGE
CONNECTICUT RIVER BASIN
MAHONEY BROOK
GARDNER, MASS.
ZONE 7
FROM TRAVERS ST. DAM DOWNSTREAM
TO DAM AT WINTER ST.
1962 PRICE LEVEL
STAGE IN FEET REFERENCED TO 1938 FLOOD
+3
+2
+1
0
1
2
-3
-4
1938 FLOOD CREST
SCALE "A"
SCALE "B"
20 40 60
0 80 160 240 320 400 480 560 640 720 800
SCALE "B"
SCALE "A"
TOTAL RECURRING DAMAGE IN $1000 UNITS

PLATE NO. B-1



	RANGE "A" 1□" = $160			RANGE "B" 1□" = $800			RANGE "C" 1□" = $10			TOTAL ANNUAL	
	AREA	LOSS	BEN.	AREA	LOSS	BEN.	AREA	LOSS	BEN.	LOSS	BENEFIT
NATURAL	13.56	$2170		4.18	$3345		0.34	$5		$5520	
MODIFIED	4.00	$640	$1530	0.96	$770	$2575	—	—	$5	$1410	$4110

APPENDIX C

HYDRAULIC DESIGN

APPENDIX C

HYDRAULIC DESIGN

1. GENERAL

As a large part of the damages experienced in 1936 and 1938 were attributed to the effect of dam failures, the strengthening and reconstruction of dams are designed to pass the standard project flood without overtopping, and in the case of Wrights Dam, to pass the spillway design flood with one foot of freeboard. Other improvements are largely influenced by existing structures that economically and practically limit the scope of feasible work. Hence the degree of protection provided by channel improvement varies in different localities, but the limiting conditions in one reach generally will not affect the protection provided in adjacent sections.

2. UPPER RESERVOIR

Following failure of the dam in 1938, an ungated 48-inch diameter culvert and a gated 48-inch diameter culvert was installed through the dam to provide additional discharge capacity. This was an improvement over the original structure, but two additional 72-inch by 44-inch pipe arch culverts will be installed through the dam. The three ungated culverts will pass the standard project flood with a total discharge of 415 cfs. The reservoir will rise 6.2 feet to elevation 1075.2 which leaves 2.4 feet of freeboard to top of dam at elevation 1077.6. A flood greater than the SPF could produce overtopping and failure of the dam, but as discussed in the following paragraphs concerning reconstruction of the Wrights Dam, the released waters would be contained in Wrights Reservoir.

3. WRIGHTS RESERVOIR

a. General. The proposed reconstruction of Wrights Dam will increase the storage capacity of the reservoir so as to make it useable for flood control. It will also provide protection against possible failure by overtopping of the existing dam.

b. Storage Capacity. The flood control storage capacity which can be provided economically in Wrights Reservoir is limited by other water uses and by improvements adjacent to the reservoir. The water level in the reservoir cannot be lowered below elevation 1065 feet, msl. without seriously affecting its value for recreation and wildlife. With spillway at elevation 1070, the flood control storage capacity will be 850 acre-feet, or about 5.1 inches of runoff. The area and capacity curves are shown on Plate 14.

c. Spillway. The spillway, having a crest length of 60 feet, was designed to pass a spillway design flood. Presumably the dam at Upper Reservoir would overtop and fail during a spillway design flood. However, failure of this dam, with Wrights Dam raised, would not be disastrous. The water level in Upper Reservoir would fall a foot or two and raise Wrights Reservoir until the two bodies of water acted like a single reservoir. To simplify the routing computations, it was assumed that inflow to Upper Reservoir would be passed unmodified into Wrights Reservoir. The flood was routed through the reservoir, starting with the reservoir full with the outlet inoperative. The maximum surcharge was 5.0 feet. The shape of the upstream face of the spillway is shown on Plate 5. The shape of the crest is the standard shape for high ogee spillways shown in EM 1110-2-1603, Figure 1, page 6. The design head is 5 feet. Discharge coefficients were determined from EM 1110-2-1603, plate 6. The rating curve is shown on Plate 14.

d. Freeboard. Because of the magnitude and character of the project, a freeboard of one foot on top of the maximum surcharge of the spillway design flood is considered adequate. The top of dam is therefore selected at elevation 1076.0.

e. Stilling Basin. Because of erodible material in the discharge channel a stilling basin will be provided. The apron will be at the level of the existing stream bed, elevation 1058.0 feet, with an end sill at elevation 1062.0. The length of the basin (20 feet) was determined from "Design of Small Dams", Bureau of Reclamation, Figure 206, page 296.

f. Outlet. Criteria for the design of the outlet are as follows:

(1) It should pass normal flows with use of but a small portion of storage capacity of the reservoir.

(2) It should be sized to obtain the optimum reduction in flood flows downstream.

(3) It should empty the reservoir within a reasonable length of time after a flood.

(4) It should maintain a permanent pool.

(5) It should provide for drawing down the permanent pool if necessary.

The outlet will be a 27-inch reinforced concrete pipe, invert elevation 1060.0 feet, through the concrete ogee spillway. A weir at elevation 1065.0 feet, with an effective length of 16 feet, will maintain the permanent pool. A 24" by 24" gate, sill elevation 1060.0 will be provided to draw down the pool if necessary. The capacity of the outlet, with the pool full to spillway crest, will be 78 cfs. The outlet will evacuate a full pool in Wrights Reservoir, plus similar storage in the upstream ponds, in about 11 days.

g. Standard Project Flood. The effectiveness of Wrights Reservoir was tested with the standard project flood (see paragraph 24). The SPF inflow to Wrights Reservoir was obtained by determining the local perimeter flow and adding the discharge from the Upper Reservoir, and the instantaneous equivalent inflow of the rainfall falling on the water surface. The total inflow was routed through the surcharge to obtain the maximum reservoir stage and the outflow. The results of the routing, shown on Plate 14 indicate a maximum pool at elevation 1071.2, and a combined spillway and outlet discharge of 335 cfs. Although an appreciable spillway discharge is produced by the SPF, the discharge occurs on the recession side of the flood hydrograph on Mahoney Brook and does not contribute to the peak.

4. WAYSIDE DAM

In order to prevent failure of the existing Wayside Dam by overtopping, additional spillway capacity will be provided to pass the peak discharge (510 cfs) of the standard project flood.

Three 72-inch by 44-inch bituminous coated corrugated metal pipe arches are being provided. A concrete U-shaped weir and stoplog structure at the entrance to one of the pipes will maintain the pond level at the present elevation. The other two pipes will each have a special sheet metal flared end section with lip elevation 1069. The three pipes together will pass the standard project flood with a maximum pool elevation of 1072.1 feet. The top of the earth dam will be raised to elevation 1073.0, thus providing a freeboard of about one foot.

5. CHANNEL IMPROVEMENTS

a. Mahoneys Pond to Chelsea Street. The channel of Mahoney Brook will be improved from Mahoneys Pond downstream to Chelsea Street. A low weir 45 feet long, crest elevation 1050.0 feet, will be constructed to maintain Mahoneys Pond at its present level. The channel below the weir will converge gradually from a bottom width of 45 feet to a bottom width of 10 feet, which will be maintained to the downstream end of the improvement. The side slopes will be 2 horizontal on 1 vertical. The bottom will slope uniformly from the toe of the weir to the invert of the Chelsea Street culvert. Dikes will be constructed along the left bank of the channel. Plate 9 shows a plan and profile of the proposed improvement.

The channel is designed to pass the peak discharge, 700 cfs of the standard project flood as modified by reservoirs as illustrated on Plate 14. The bottom slope of the channel will be about 1.8 percent. Velocity will be supercritical (10.4 ft/sec) except where affected by backwater.

There will be three hydraulic control sections. The first control will be the weir at the outlet of Mahoneys Pond. The present outlet has a capacity of only 240 cfs before water begins to flow over the earth dike. The proposed weir will pass the peak discharge, 700 cfs of the standard project flood with a surcharge of 2.6 feet.

The next control section will be at the downstream end of the transition. Below this point the channel will be steeper than critical slope.

The third hydraulic control will be the culvert under Chelsea and Summer Streets at the downstream end of the channel improvement. It is 10 feet by 6 feet. The entrance will be improved by the removal of the ell of a building together with its foundation walls. The approach channel will be deepened to the grade of the culvert, and new wing-walls will be constructed. The present capacity of the opening under the building ell is only 300 c.f.s. This will be increased to safely pass 600 c.f.s. by the proposed improvement. Plate 9 shows the computed flow line for the standard project flood of 700 c.f.s.

b. Quality Pad Company. The improvement will consist of the removal to elevation 1025.0 of the remains of the old gate structure at the Quality Pad Company dam downstream of Summer Street. The existing structure has a discharge capacity of only 240 cfs before the water begins to overtop the dam. The channel just downstream of the gate structure is about 12 feet wide with rubble masonry walls. The discharge capacity will be increased to approximately 420 cfs.

c. Boston & Maine Railroad Bridge. The complex of the South Main Street bridge and the Boston & Maine Railroad bridge comprises a serious restriction to major flows. The angle of the railroad bridge abutments, immediately downstream of Main Street bridge, partially impedes flow and causes it to make an abrupt turn. Utility pipes suspended through the highway bridge, and low steel in the railroad bridge produce further restrictions that will likely catch debris during floods. Local dumping into the channel at the right railroad bridge abutment has also reduced the cross-sectional area. Reconstruction and realignment of the bridges are desirable hydraulically but the cost of such improvements would be prohibitive. The only feasible improvement is removal of the dumped material below the railroad bridge and construction of a retaining wall to hold the bank. This will increase the cross-sectional area at a critical area and help deflect flow back into channel. The work at the railroad bridge, plus the removal of the remains of a dam at Travers Street will improve the stage-discharge relationship upstream of the two bridges. Assuming some impeded debris, it is likely that some damage would be experienced during a standard project flood in the area at the junction of Mahoney and Foster Brooks. As may be deduced from the profiles on Plate 4, backwater from this section of the brook will not affect other improvements on Mahoney and Greenwood Brooks.

d. Dam at Traverse Street. The remains of a partially destroyed dam, located just upstream of Travers Street and about 200 feet downstream of the railroad bridge described in 38-d, will be removed. Complete removal of the dam will lower tailwater depths below the railroad bridge, and hence improve the discharge rating curve of the bridge complex at South Main Street. Removal of the dam will also improve flow conditions approaching the Travers Street Bridge.

APPENDIX D

FOUNDATIONS, EMBANKMENT AND MATERIALS

APPENDIX D

FOUNDATIONS, EMBANKMENTS AND MATERIALS

a. Foundation Conditions.

(1) Detailed geologic reconnaissance and a program of six (6) foundation borings and twenty-nine (29) hand probings have been completed to determine foundation characteristics for final design. The locations of all foundation explorations are shown on Plate No. 11. All foundation borings were made by continuous drive-sampling methods in overburden for recovery of 1½-inch minimum diameter samples. Where bedrock was important to design, or was encountered at shallow depths, it was diamond-drilled for recovery of 2-1/8-inch diameter cores. All soil samples obtained from the borings were visually classified according to the Unified Soil Classification System, supplemented as necessary by gradation tests. All rock cores taken from the borings were inspected and logged. Detailed classification and description of materials encountered in each of the foundation borings, and a table of probing data are shown on Plate No. 12. A summary of laboratory test results is shown on Plate No. 5-1.

(2) Reconnaissance and explorations in the structure areas indicate the overburden materials to consist principally of glacial till, overlain here and there by limited amounts of outwash and/or swamp deposits, part or all of which are in some places capped by substantial sections of artificial fill. These materials have a combined thickness of five (5) to twenty (20) feet. Though differing in mode of origin, the till, outwash and fill materials are similar in character and are often difficult to distinguish from one another. This is particularly true of the materials at and near the surface.

(3) The glacial till, which rests on bedrock, is widespread in occurrence and, because it is present both at the surface and at depths of generally less than 10 feet at most of the structure locations, it constitutes the material upon which most of the structures will be founded. These materials range from 3 to 10 feet in thickness and consist largely of moderately compact, to compact gravelly silty sands, (SM) and silty sandy gravels (GM), with numerous cobbles. The outwash materials are rather limited in occurrence, being present locally at some of the structure locations and totally absent at others. These materials overlie the till in a variably thin section, 1 to 10 feet, and thereby constitute a small portion of the foundation materials. The composition of these materials is quite similar to that of the till except that their silt content is slightly lower and the degree of compaction slightly less. For this reason the sands are classed as (SP-SM) and the gravels as (GP-GM). Foundations for embankments and structures will not be taken to rock except for the concrete spillway at the Greenwood Brook site.

(4) Deposits of soft muck, from 0.5 to 3.0 feet in depth, constitute the surface materials in the bottom of the ponds and in the swampy areas. This material consists mainly of organic silts. As indicated by hand probings made along some of the structure alignments, these deposits overlie firm or compact material consisting probably of sands and gravels. Thin 1-foot thick sections of silt (ML) were encountered between the artificial fill and the natural deposits in foundation borings made at the Wayside Pond and West Dike (Wright's Reservoir) structure locations.

(5) The artificial fill materials, which cap the natural deposits at some locations, consist principally of silty sand, silty sandy gravel and gravelly silty sand, all of which contain numerous cobbles. Cinders and other debris are also present in the fill materials at a few locations. These fill materials range generally from a few feet to over 10 feet in thickness, heaviest accumulations being present in the existing dikes.

(6) Bedrock, besides being exposed along the west side of Wright's Reservoir, was encountered in foundation explorations at depths of 8 to 18 feet, elsewhere in the project area. These rocks are comprised of granite, pegmatite and granitized schist. At the location of the proposed concrete spillway structure on Greenwood Brook, bedrock was encountered in borings FD-4 and FD-5 at depths of 14.1 and 18.6 feet respectively. Here the rock is a granitized schist and, though it is highly jointed and fractured, is adequate for proposed structure foundations. Bedrock was also encountered at a depth of 13.6 feet in boring FD-6, located near the downstream end of the proposed conduit at the Wayside Pond structure. This rock is a coarse-grained, much altered and mineralized granite, yet has adequate foundation properties. Granitic bedrock was encountered in borings FD-2 and FD-3 at depths of 8.7 and 15.1 feet respectively, along the southerly and central reaches of the proposed Wright's Reservoir - West Dike. This rock will not be involved in structure foundations however. It is not expected that bedrock will be encountered in channel improvement excavations between the Mahoney Brook spillway location and Chelsea Street, downstream.

(7) The general level of subsurface water is high in the project area, ranging from a depth of five (5) feet in areas adjacent to the ponds and brooks, to or slightly above ground surface in the swampy areas. This high level is effected by the presence of impervious till materials and bedrock at shallow depths, and the numerous ponds and brooks in the area.

b. Availability of Construction Materials.

Because of the wide distribution of the structure locations on this project and, in view of the small quantities of required embankment materials, no explorations were made for borrow and borrow sources have not been established. All types of random materials will be obtained from required excavations, except for a limited amount for embankments at the north end of Wright's Reservoir, which materials will be furnished by the contractor from developed or undeveloped sources of which there are a number within a short haul distance of the structure location. Gravel for fill and bedding material may be obtained commercially from any of four sources within a 15-mile haul distance of the project. Protection stone is available from three commercial sources within a 15-mile radius of the project.

Concrete Aggregate. Approximately 300 cubic yards of concrete will be required for construction of spillway weir and buttress walls. In view of the small quantity of concrete required, aggregate investigation has been confined to established commercial sources supplying transit-mix concrete plants within a 20-mile haul distance of the site. R. T. Curtis, Incorporated, Barre, Massachusetts, has previously been tested and approved as a fine and coarse aggregate source for other Corps of Engineers projects. Data on this source are reported in Technical Memorandum No. 6-370 "Test Data - Concrete Aggregate in Continental United States," Volume 5. Latitude 42 - Longitude 72, Index No. 9. The processing plant for sand and gravel and concrete batching plant are located in Barre, Massachusetts, a 14-mile haul distance from the project site. Curtis also operates a concrete plant located in Templeton, Massachusetts, a haul distance of 4 miles from the project site. P. J. Keating Company, with plants for processing sand and gravel and crushed quarry stone and for batching concrete, are located in Lunenburg, Massachusetts, a haul distance of 20 miles from the project site. Keating also operates a concrete batching plant located in Fitchburg, Massachusetts, a haul distance of 10 miles from the project site. Concrete materials from this source have been used on many city and state projects in recent years, as well as Corps of Engineers projects. The size of the project and the quantity of concrete does not warrant evaluation testing of concrete aggregate sources. Approved sources of aggregates will not be listed in the specifications. If the contractor elects other than a previously approved source, evaluation testing will be performed on the source submitted.

c. Characteristics of Foundation Materials.

(1) General. A subsurface investigational program conducted to obtain data for design and engineering studies is described in paragraph 1, "Site Geology." A general description of the foundation materials is given in paragraph 2, "Site Geology."

(2) Description and Distribution of Materials.

(a) Dike and Spillway Structure (Wright's Reservoir). The overburden materials in the foundation areas of the dike and spillway at the north end of Wright's Reservoir consist mainly of a variable natural deposit of firm non-stratified soils overlain, within the present reservoir limits by a surficial deposit of organic silt, and overlain at the east end of the spillway foundation area by man-made existing dike fill. The pertinent soils in the natural firm overburden deposit are somewhat variable and consist of moderately compact to compact silty sandy gravel (GP-GM) gravelly silty sand (SM) and silty sandy gravel (GM), all containing numerous cobbles and boulders. In general, the silt contents of the soils in this deposit vary between 10 and 35 percent by weight of the components of the materials passing the Number 4 U. S. Standard Sieve. The limits and locations of the various gradations of soils were not determined. The man-made dike fill in the area at the east end of the spillway varies in thickness up to 11 feet and consists mainly of compact silty sandy gravel containing numerous cobbles. The surficial deposit of organic silt within the limits of the reservoir has a thickness of 1 to 3 feet and is very soft. The bedrock surface at the east and west ends of the spillway area is at a depth of 18.6 feet (about elevation 1,052) and at a depth of 14.1 feet (about elevation 1,050) respectively. The topsoil in the land area is generally less than 1 foot in thickness.

(b) West Dike (Wright's Reservoir). Overburden materials in the foundation area of the dike at the west end of Wright's Reservoir consist mainly of a firm non-stratified natural soil deposit overlain from about Sta. 1+50 to about Sta. 5+00, by fill materials of an existing road. The natural firm soils are somewhat variable and consist of compact gravelly silty sand (SM) and silty sandy gravel (GM), both of which contain numerous cobbles. The silt contents of the soils in this deposit vary between 15 and 35 percent by weight of the components of the materials passing the Number 4, U. S. Standard Sieve. The limits and the locations of the various gradations of the soils were not determined. The road fill material varies in thickness up to about 8 feet at the location of FD-3 and consists mainly of moderately compact silty sand and silty sandy gravel. The bedrock surface was encountered at a depth of 8.7 feet (about Elevation 1,060) at the south end of the alignment and at a depth of 15.1 feet (about Elevation 1055) at the center of the alignment. Bedrock was not encountered at the north end of the alignment to an explored depth of 10 feet or about elevation 1,060. The topsoil in the area is generally about 1 foot in thickness.

(c) Dike and Conduit Structures (Wayside Pond). Overburden materials in the foundation areas of the dike and conduit structures at the west end of Wayside Pond consist mainly of firm non-stratified natural soils, overlain, within the present pond limits by a surficial deposit of 1 to 3 feet of very soft organic silt and overlain at the north and south end and along the landside toe of the dike by road fill materials. Natural overburden materials are variable in character consisting of compact gravelly silty sand (SM) and silty sandy gravel (GM) containing numerous cobbles. The silt contents of the materials vary between 15 and 35 percent by weight of the components of the material passing the Number 4, U. S. Standard Sieve. The limits and locations of the various gradations of the soils were not determined. The road fill material along the landside toe of the dike and at the north and south end of the dike varies in thickness up to about 4 feet and consists of silty sandy gravel mixed with rubble and coal particles. The surficial deposit of organic silt within the limits of the pond has a thickness of 1 to 3 feet and is very soft. The bedrock surface was encountered at a depth of 13.6 feet or about elevation 1,059.

(d) Dike and Spillway Structure (Mahoney Pond). The overburden materials in the foundation areas of the dikes and of the spillway structure at the west end of Mahoney Pond consist mainly of firm non-stratified natural soil overlain, within the pond limits by 1 to 4 feet of very soft organic silt and overlain, at the extreme west end of the dike west of the spillway structure and in the reach of the dike south of the spillway structure by man-made fill. Foundation drive sample borings were not considered necessary at either the spillway or dike alignments because of the very low heights of the structures. Site reconnaissance and general geology of the area indicate that the firm foundation materials underlying the very soft organic silt consist probably of silty sandy gravel (GM) and gravelly silty sand (SM) having silt contents of from 15 to 35 percent of the components of the materials passing the Number 4 U. S. Standard Sieve. The man-made fill in the land area at the extreme end of the dike west of the spillway structure varies in thickness up to about 3 feet and consist mainly of loosely dumped sawdust and wood shavings. The man-made fill in the land area of the dike south of the spillway structure varies in thickness up to about 5 feet and consists mainly of coal clinker, wood shavings, sand and gravel, bordered at the waters edge by numerous large boulders. The topsoil in the area is generally about 1 foot in thickness.

(e) Shear Strengths. No shear tests were performed on samples of foundation materials for this project. On the basis of visual examination of the samples and their grain size distribution curves, the exploration logs and experience with similar materials, the following minimum values of angles of internal friction and cohesion are estimated for the various types of foundation materials:

(1) All materials except man-made fills and organic silt in the pond areas - $\phi = 30^{\circ}$, $c = 0.0$ psf.

(2) Organic silt in pond areas - $\phi = 0^{\circ}$, $c = 200$ psf.

(3) Man-made fills (except for rubbish, wood shavings and other unsuitable material) - $\phi = 25^{\circ}$, $c = 0.0$ psf.

(f) Permeabilities. Considering the size and shape of the structures, the low hydraulic heads, and the homogeneous character of the foundation materials, the permeability characteristics of the foundations are not considered to be of major significance except at the West Dike at Wright's Reservoir. At the west dike of Wright's Reservoir, the road fill materials consist of silty sandy gravel at the location of FD-3.

For purposes of design, however, it is assumed that the materials vary throughout the fill area, and it is estimated that the permeability of the material will also vary from about 100 to 300 x 10^{-4} cm/sec. The permeability of the firm non-stratified material soils below the fill, based on visual examination of the samples and their grain size curves, varies from about 1 to 50 x 10^{-4} cm/sec.

(g) Consolidation Characteristics. No consolidation tests were performed on samples of foundation materials for this project. Except for the organic silt, these materials exhibit very low compressibilities. The organic materials, while relatively compressible, occur generally in thin strata and their consolidation will be of minor magnitude.

d. Design of Embankments.

(1) Criteria. Current design criteria as set forth in the pertinent sections of the Engineering Manual for Civil Works Construction have been followed in the design of the embankments for this project.

(2) Selection of Embankment Sections. Typical embankment sections for this project are shown on Plate Nos. 5, 6, 8, and 9. The embankment sections selected for the dikes have been developed from investigations and studies of foundation conditions and of the characteristics of available construction materials and foundation soils. The selected dike sections (except for the dike south of the spillway structure at the west end of Mahoney Pond) are essentially of the homogeneous rolled-earth fill type except in pond areas below an elevation 2 feet above existing water surface, where the embankment material will consist of either dumped gravel fill or dumped random fill. The dike south of the spillway at the west end of Mahoney Pond will be composed entirely of dumped random fill. The dikes will have seeded topsoil on the land slopes and on that portion of the pond slopes above water, except for an area immediately adjacent to the inlets of the conduit structures at the Wayside Pond Dike where stone slope protection will be used on the pond side slope. Embankment heights are generally less than 6 feet on the land side except at the West Dike at Wright's Reservoir where the maximum height of dike is upwards of 16 feet. Embankment heights on the pond side are generally under 6 feet except for the Dike at the north end of Wright's Reservoir where the maximum height of dike approaches 11 feet. Side slopes of 1 vertical on 2 horizontal have been adopted for all land dikes and for those portions of pond dikes to be constructed above an elevation 2 feet above existing water surfaces. For the portions of dikes to be constructed below water, to provide a firm foundation for the overlying dike, the fill will be constructed with a 15-foot berm and a 1 on 3 slope at the north end of Wright's Reservoir, and a 5-foot berm and a 1 on 3 slope at the West end of Mahoney Pond. The wrap-around sections, where the dikes butt up against the spillway structures at the north end of Wright's Reservoir and at the west end of Mahoney Pond, will be composed of compacted

impervious fill. Erosion control at the wrap-around toes will consist of stone slope protection over gravel bedding extending into the river channel.

(3) Characteristics of Embankment Fill Materials. Embankment fill materials will be obtained from required excavations and from approved sources furnished by the contractor. Impervious fill material to be furnished by contractor will consist of non-plastic glacial till containing from 75 to 95 percent of particles passing the Number 4 U. S. Standard Sieve and at least 25 percent passing the Number 200 U. S. Standard Sieve. It is estimated that the impervious fill material in place will have a coefficient of permeability of 1×10^{-4} cm/sec and will develop an angle of internal friction of at least 30 degrees. Gravel fill material to be used in those portions of the embankment placed in water will be furnished by the contractor and will consist of gravelly sand or sandy gravel containing from 40 to 70 percent of particles passing the Number 4 U. S. Standard Sieve and no more than 15 percent of the fraction passing the Number 4 U. S. Standard Sieve shall pass the Number 200 U. S. Standard Sieve. It is estimated that this material if placed without compaction will have a coefficient of permeability of from 20 to 100×10^{-4} cm/sec. Random fill will consist of five different types of materials and will be obtained either from required excavations or from approved sources furnished by the contractor, as discussed hereinafter. Random fill, Material A, to be furnished by the contractor, and random fill, Material B, to be obtained from required excavations for the spillway structure at the north end of Wright's Reservoir will be used to form a foundation for embankments within the reservoir limits, and as fill for the spillway structure and will consist of non-plastic granular material free of debris, stumps, topsoil, and organics. Random fill, Material C, to be obtained from the required excavation for the precast concrete crib wall at South Main Street will be used as dumped fill behind the crib wall structure and will be free of debris, stumps, topsoil, and organics. Random fill, Material D, will be obtained from the required excavation for the pipe conduits at Upper Wright's Reservoir and will be used as fill around and above the pipe conduits and will be free of debris, stumps, road pavement, topsoil and organics. Random fill, Material E, will be obtained from the required excavation for the channel improvement west of the spillway structure at the west end of Mahoney Pond and will be used in the embankment southwest of the spillway structure. The material will consist of solid fill and will not include clearing and grubbing material or nested boulders. It is estimated that each of the five types of random fill material in place will have a coefficient of permeability of from 5 to 100×10^{-4} cm/sec. and each will develop an angle of internal friction of at least 25 degrees.

(4) Control of Seepage. In view of the very small hydraulic heads acting on the dike embankments and of the low permeability of the impervious fill and non-stratified foundation materials, special

provisions for control of seepage through the embankments and foundations is not required except at the West Dike of Wright's Reservoir. In the roadway reach of the West Dike, the road fill materials are relatively pervious (based on samples taken at the location of FD-3), and detrimental dike foundation seepage could occur if not properly controlled. To control seepage through the embankment foundation in this reach, a zone of gravel fill has been incorporated into the landside toe of the embankment section.

(5) Slope Stability. Slopes of 1 on 2 for the land embankment sections are considered to be adequately safe against shear failure because of the low embankment heights and presence of firm foundation materials. Slopes of 1 on 2 have also been adopted for the embankment section extending across Wayside Pond where 1 to 3 feet of very soft organic silt is present over the firm foundation materials. These slopes are considered adequate because the embankment is low in height and will not be subjected to any hydraulic head, and it is anticipated that most of the very soft organic silt will be displaced when the embankment fill materials are placed. In other reaches of dike alignments that extend across ponds containing surficial deposits of 1 to 4 feet of very soft organic silt and are subjected to hydraulic heads up to about 8 feet, slopes of 1 on 2 have been adopted for the land dikes to be constructed above an elevation 2 feet above existing water surface and for the portions of the dikes to be constructed below water which will serve as a foundation for the overlying embankments. A 15-foot berm with a slope of 1 on 3 has been adopted at the north end of Wright's Reservoir, and a 5-foot berm with a 1 on 3 slope at the west end of Mahoney Pond. Although these embankments will be constructed on the soft deposit of organic silt, the slopes provided are considered adequate because of the low heights of the embankment and because it is planned to push out most of the very soft organic silt to the front of the waterside toe of dike.

(6) Settlements. It is anticipated that, in general, settlement of the embankment foundation and the embankment materials will not be significant and will occur during construction.

(7) Slope Protection. Since it is anticipated that the water side slopes of the dikes will not be subjected to any significant wave, seeded topsoil is considered adequate to provide protection against erosion. Stone slope protection, 12 inches in thickness, over 12 inches of gravel bedding is provided immediately adjacent to the inlets of the conduit structures where eddying action could occur. The stone slope protection will be reasonably well-graded from a minimum size of stone of approximately 2-inch diameter to a maximum size of approximately one cubic foot. Average size of stone will be between 30 and 50 lbs. with diameter between 8 and 10 inches, respectively. Maximum size of stone will not exceed 150 lbs. Landside slopes will also be topsoiled and seeded to provide protection against erosion.

(8) Construction Considerations.

(a) Removal of Unsuitable Foundation Materials. Removal of unsuitable foundation materials will be limited to the stripping of topsoil and the excavation of man-made fill at the extreme end of the dike west of the spillway structure at the west end of Mahoney Pond.

(b) Embankment Construction in Water. As indicated on the typical embankment sections, embankment fill to be placed in water beneath the pond side slopes of the dikes will consist of dumped gravel fill and dumped random fill having the characteristics described previously in this report. Dumping of fill materials will commence at the edge of the ponds and will progress towards the center of the ponds in order to push the very soft organic silt to the front of the waterside toe of the dikes so that it will not become trapped in the foundation areas. In order to provide a trafficable surface, the material will be dumped to an elevation 2 feet above present water levels.

(c) Spoil Area. No spoil area will be provided. Spoil material, to be spoiled off the project site, will become the property of the contractor.

e. Design of Channels.

(1) Slope Stability. Explorations to determine the character of overburden materials, were not made along the channel alignment West of the Mahoney Pond spillway and along the channel alignment north of the Wright's Reservoir spillway. Site reconnaissance and general geology of the area indicate that the overburden materials are firm and consist probably of silty sandy gravel (GM) and gravelly silty sand (SM) having silt contents of from 15 to 35 percent of the components of the materials passing the Number 4 U. S. Standard Sieve. The topsoil cover in the channel areas is about 1 foot in thickness. Channel side slopes of 1 on 2 have been adopted and are considered sufficiently adequate for the 5 to 6-foot maximum height slopes based on previous experience at other sites with similar overburden materials.

(2) Slope Protection. Estimated velocity in the Mahoney Pond channel and in the Wright's Reservoir channel are 10 and 6 ft/sec., respectively, which require stone slope protection along both sides and bottoms of the channels. The stone slope protection will have a minimum thickness of 12 inches and will be laid on a 12-inch thickness of gravel bedding in order to prevent undercutting. The stone slope protection will be reasonably well-graded from a minimum size of stone of approximately 2-inch diameter to a maximum size of approximately one cubic foot. Average size of stone will be between 30 and 50 lbs., with diameter between 8 and 10 inches, respectively. Maximum size of stone will not exceed 150 lbs.

(3) Foundation Seepage Control for Concrete Structures. The spillway structure at the north end of Wright's Reservoir will be founded on bedrock and detrimental foundation seepage below the structure will not occur. To prevent detrimental seepage around the ends of the wingwalls, a wrap-around section having a creep ratio of more than 6 has been provided. The spillway structure at Mahoney Pond will be constructed on natural firm non-stratified relatively impervious materials and will be subjected to a maximum hydraulic head of about 2 feet. In view of the non-stratified character of the foundation soils, the low hydraulic head, and a provided creep ratio greater than 6, the structure is considered safe against failure due to seepage.

(4) Foundations for Concrete Structure. The foundation conditions for the concrete spillway structures are described and discussed in Paragraph 1, above. The control of foundation seepage is discussed in Paragraph 4, above. The foundation grade for the spillway structure at Mahoney Pond has been established at such an elevation as to provide a silty sandy gravel or gravelly silty sand foundation of sufficient bearing capacity to safely sustain the anticipated loadings. The concrete spillway structure at the north end of Wright's Reservoir will be constructed on bedrock.

(5) Seepage Control Along Embedded Pipes. The pipe conduits at the west end of Wayside Pond and at Upper Wright's Reservoir will consist of 72" x 44" Bituminous Coated Corrugated Metal Pipe Arches. The hydraulic head acting on the pipe conduits at the west end of Wayside Pond could be upwards of about 6 feet, and detrimental seepage could occur along the contact of the corrugations of the pipe and the fill material, if not properly controlled. To control the seepage along the pipe, a concrete slab 1 foot thick, for a distance of 20 feet at the waterside end, has been provided along the bottom and along the sides of the pipes up to the spring line. The concrete will provide a tight seal between the corrugations and the fill material. To control seepage along the sides of pipes, specification will provide for the placing of fine materials immediately adjacent to the pipes so that corrugations will be completely filled. Detrimental seepage along the pipe conduits at Upper Wright's Reservoir is not expected to occur because of the very low hydraulic heads and long path of seepage.

APPENDIX E

STRUCTURAL DESIGN

COMPUTATIONS

PLATE

APPENDIX E

STRUCTURAL DESIGN

a. Purpose. - This section of the design memorandum presents the design criteria, basic data and assumptions used in the structural design of the appurtenant structures. A brief description of the structures with loading conditions and assumptions used is included to show the design procedures. Typical computations are included in the Appendix showing the maximum conditions for the critical structures.

b. Scope. - The structural design of the spillway retaining walls, spillways and crib wall is included herein.

c. Design Criteria. -

(1) General. - All working stresses conform to those specified in the Engineering Manual EM 1110-1-2101, "Working Stresses for Structural Design," dated 6 January 1958. Loading conditions, design assumptions and other design criteria are based on the following applicable parts in the Engineering Manual for Civil Works; Standard Practice for Concrete (Part CXX, October 1953), Structural Design of Spillway and Outlet Works (Part CXXIV, Dec. 1952) and Retaining Walls (EM 1110-2-2502, dated 29 May 1961). Accepted engineering practice has been employed in cases where the Engineering Manual for Civil Works does not apply.

(2) Concrete. - The following table lists the concrete and reinforced concrete stresses used in the design of structures.

(1) Structures Other Than Concrete Pipe. -

Flexure	Lbs. Per Sq. In.
Extreme fiber stresses in compression	1,050
Extreme fiber stresses in tension (plain concrete)	60
Shear (v) -	90
Bond-(u) Deformed Bars -	
Top bars	210
All others	300
Modular Ratio-(n) -	10

(3) Reinforcement. -

(a) Grade and Working Stresses. - All reinforcement in the structures including temperature and shrinkage reinforcement was designed for the working stresses of new billet steel, intermediate grade, deformed bars which is 20,000 p.s.i. in flexural tension. The reinforcement will conform to the requirements of Federal Specification QQ-S-632, Type II, Grade C and to ASTM A-305-56T.

(b) Minimum Cover for the Main Reinforcement. - The minimum cover from main steel reinforcement to surface was maintained at 3" except for bottom of base slabs which will be 4".

(c) Splices. - All splices will be lapped 24 diameters to develop by bond, the total working strength of the bars.

(d) Temperature and Shrinkage Reinforcement. - Temperature and shrinkage reinforcement will be provided where the main reinforcement extends in only one direction. Such reinforcement will provide for a ratio of steel area to concrete area (bd) of 0.002 with a minimum of .0012 in each face up to a maximum of #6 bars at 12" cc.

d. Basic Data and Assumptions. -

(1) Controlling Elevations of Dam and Appurtenant Structures (m.s.l.). -

(a) Wrights Reservoir. -

Top of Dam	1076.0
Spillway Crest	1070.0
Maximum Water Surface just upstream at Spillway Weir	1075.0

(b) Mahoney Pond

Top of Dam	1055
Spillway Crest	1050
Maximum Water Surface just upstream at Spillway Weir	1053

(2) Loads. -

(a) Dead Loads. - The following unit weights for materials were used:

MATERIAL	UNIT WEIGHT (Lbs/cu.ft.)			
	Dry	Saturated	Moist	Submerged
Earth fill	130	140	140	82
Concrete (plain & reinforced)	150			

(3) External Water Pressure. - Triangular distribution of the water pressure in the reservoir pool on the structures was used. Tailwater pressure was taken at 60% of full value for the spillway section.

(4) Internal Water Pressure. - Uplift pressure under structures was assumed effective on 100% of the area of the base, varying uniformly from tailwater head at the toe to full headwater at the heel.

(5) Earth Pressure. - Earth pressures were determined in general accordance with EM 1110-2-2502, Retaining Walls, dated 29 May 1961. "At rest" pressures were used in all cases.

(6) Earthquake Forces. - Because of the small size of the structures involved, earthquake forces are not a factor and were disregarded.

(7) Ice Pressure. - Horizontal forces due to ice pressure were included in the design of the weirs. A value of 6,000 lbs. per linear ft. was used in the design of the Wrights Pond Spillway. A value of 2,000 lbs. per ft. was used for the Mahoney Pond Spillway because of the upstream shape of the weir which tends to force the ice upward at the weir.

(8) Frost Protection. - On the basis of temperature records and frost penetration depth curves derived by the Arctic Construction and Frost Effects Laboratory of the Corps of Engineers, a minimum frost protective cover of 4 feet above foundation level will be used for any structures founded on earth.

e. Weir at Wrights Reservoir. -

(1) Description. - The ogee shaped weir is approximately 60 feet in length with crest elevation at 1070 and channel bed elevation at 1058. The abutments are gravity type walls. The spillway and abutments will be founded on rock. Contraction joints in the weir will be provided at an 20'-0" spacing.

(2) Spillway Stability. - The following loading conditions were investigated in the design.

Case I - Reservoir empty and dead load of weir.

Case II - Reservoir to spillway crest, no tailwater, uplift varying from full headwater at the heel to 0 at the toe.

Case III - Reservoir at El. 1066 with ice load applied.

Case IV - Reservoir at maximum surcharge elevation of 1075.0 and maximum tailwater elevation at 1066.6, uplift varying from maximum headwater at heel to maximum tailwater at toe.

(3) Spillway Design. - Maximum bearing is only 3040 psf under Case I loading and the resultant falls within the middle third of the base for all conditions of loading. Under Case IV loading the shear friction coefficient was found to be 9.6 and rock anchors will not be required.

(4) Gravity Wall. - The gravity wall at the abutments was investigated for a normal condition assuming saturated soil behind the wall and a flood condition with water to maximum surcharge elevation 1075.0 and tailwater at the base of the wall. An at rest coefficient of 0.5 was used.

f. Weir at Mahoney Pond. -

(1) Description. - The ogee shaped weir is approximately 45 feet in length with crest elevation of 1050. The spillway will be founded in earth. Gravity type walls will be utilized for each abutment.

(2) Spillway Stability. - The spillway was investigated for loading conditions similar to Wrights Reservoir spillway, except that the ice load was taken at 2000 lbs. per ft. horizontally.

(3) Spillway Design. - Maximum bearing is 1173 psf under Case I loading. The maximum lateral pressure will be resisted by passive pressure and a key has been provided on the upstream edge to develop the passive pressure. Steel reinforcement will be provided in the key only.

(4) Gravity Walls. - The gravity walls will be investigated for the construction condition; a rapid-drawdown condition assuming saturated soil and no uplift and a flood condition with water at maximum surcharge elevation of 1053. An at rest coefficient of .5 will be used.

g. <u>Crib Walls</u>. - The crib wall will be either commercially available concrete interlocking ties or metal cribbing. Stability computations are included for the concrete type cribbing.

h. <u>Metal Culvert Pipes</u>. - Wherever metal culvert pipe is indicated, it will be commercial corrugated pipe designed to carry highway or earth dike loadings in accordance with AASHO and the manufacturer's standards.

SUBJECT Gardner, Mass Local Protection Project

COMPUTATION Spillway Stability

COMPUTED BY BFW CHECKED BY DATE Sept 1962

LOADING CONDITIONS

CASE I CONSTRUCTION CONDITION

Dry
30 PSF Wind on Downstream Face
Resultant within Mid 1/3

CASE II NORMAL OPERATING CONDITION

Pool at Spillway Crest
Minimum Tailwater
No Ice
Resultant within Mid 1/3 Max $\frac{\Sigma H}{\Sigma V} = 0.65$

CASE III Ice at Normal Pool Elevation

Resultant within Mid 1/3
Max $\frac{\Sigma H}{\Sigma V} = 0.65$

CASE IV FLOOD DISCHARGE CONDITION

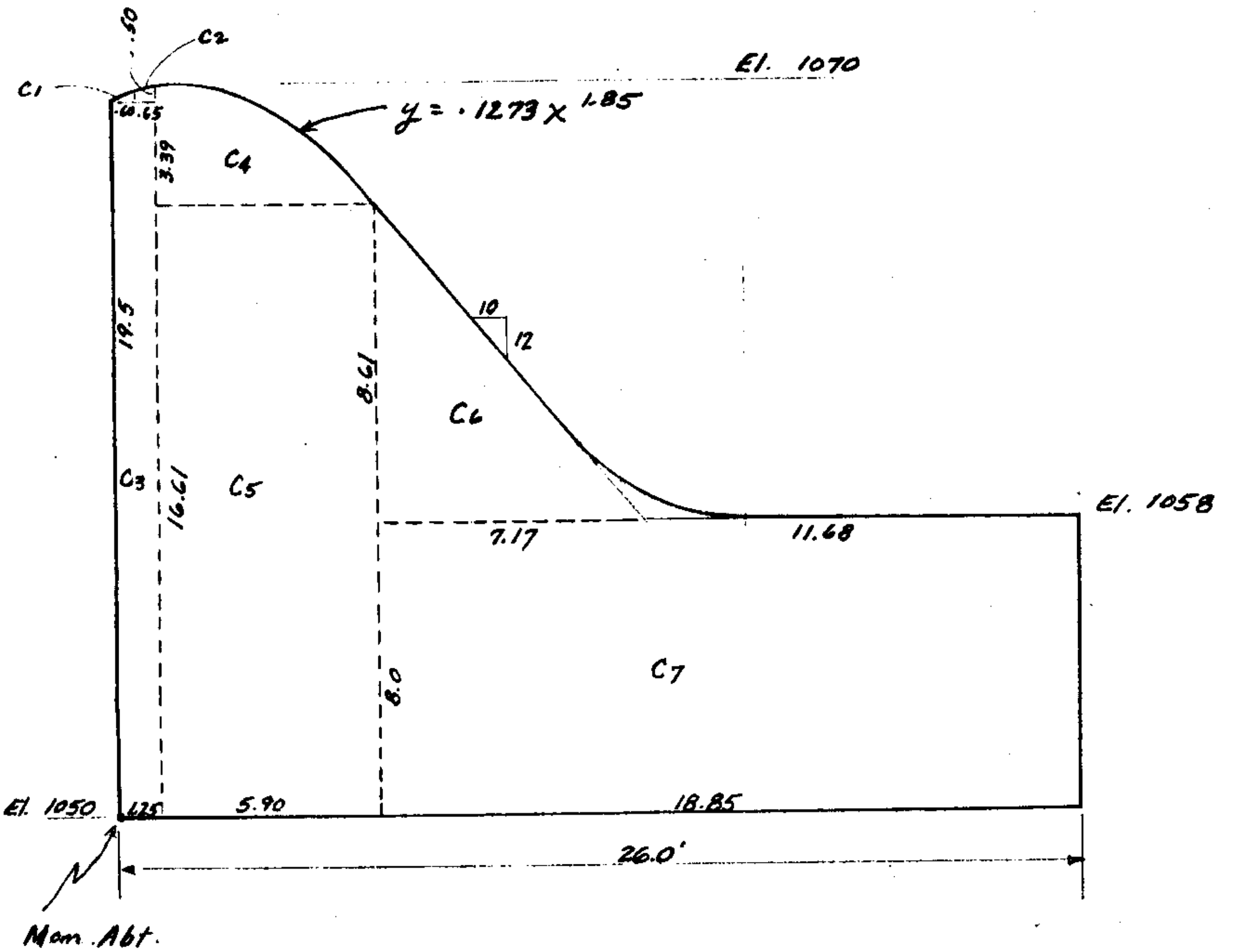
Pool at Max. Surcharge
Maximum Tailwater
T.W. at Full Value for Uplift
T.W. at 60 % Full Value for Lateral Forces
No Ice
Resultant within Mid 1/3
Max $\frac{\Sigma H}{\Sigma V} = 0.65$

Min $S_{S.F} = 4$ when $\frac{\Sigma H}{\Sigma V} > 0.65$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT [illegible], Mass. Local Protection Project
COMPUTATION Wright's Reservoir Spillway Stability
COMPUTED BY [illegible] CHECKED BY BHV DATE 19 Sept. 62



SUBJECT Gardner, Mass. Local Protection Project
COMPUTATION Wright's Reservoir Spillway Stability
COMPUTED BY BFW CHECKED BY DATE Sept 1962

NAPPE PROPERTIES OF C4 PARABOLA

$$y = 3.39 - .1273x^{1.85} \qquad dA = y\,dx$$

$$dA = 3.39dx - .1273x^{1.85}dx$$

$$A = \int_0^{5.90} 3.39\,dx - \int_0^{5.90} .1273x^{1.85}dx = \Big[3.39x\Big]_0^{5.90} - \left[\frac{.1273x^{2.85}}{2.85}\right]_0^{5.90}$$

$$A = 3.39(5.90) - \frac{.1273(5.90)^{2.85}}{2.85} = 20.0 - 7.02 = \underline{\underline{12.98^{\square'}}}$$

$$dM_y = x\,dA = xy\,dx$$

$$M_y = \int_0^{5.90} 3.39x\,dx - \int_0^{5.90} .1273x^{2.85} = \left[\frac{3.39x^2}{2}\right]_0^{5.90} - \left[\frac{.1273x^{3.85}}{3.85}\right]_0^{5.90}$$

$$M_y = 1.695(5.90)^2 - \frac{.1273(5.90)^{3.85}}{3.85} = 59.0 - 30.8 = 28.2\ ft^3$$

$$\bar{x} = \frac{M_y}{A} = \frac{28.2}{12.98} = \underline{\underline{2.17'}}$$

$$M_x = \int \frac{y^2 dx}{2} = \int_0^{5.90} (3.39 - .1273x^{1.85})^2 \frac{dx}{2}$$

$$= \int_0^{5.90} \left[\overline{3.39}^2 - 2(3.39)(.1273x^{1.85}) + \overline{.1273}^2 x^{(1.85)(2)}\right] \frac{dx}{2}$$

$$= \int_0^{5.90} \frac{\overline{3.39}^2 dx}{2} - \int_0^{5.90} 3.39(.1273x^{1.85})dx + \int_0^{5.90} \frac{\overline{.1273}^2}{2} x^{3.70} dx$$

$$= \left[\frac{\overline{3.39}^2 x}{2}\right]_0^{5.90} - \left[\frac{3.39(.1273)x^{2.85}}{2.85}\right]_0^{5.90} + \left[\frac{\overline{.1273}^2 x^{4.70}}{2 \times 4.70}\right]_0^{5.90}$$

$$= 33.9 - 23.8 + 7.24 = 17.34\ ft^3$$

$$\bar{y} = \frac{M_x}{A} = \frac{17.34}{12.98} = \underline{\underline{1.34'}}$$

SUBJECT Gardner, Mass Local Protection Project
COMPUTATION Wright's Reservoir Spillway Stability
COMPUTED BY [illegible] CHECKED BY [illegible] DATE 19 Sept. 62

[illegible] [illegible] [illegible]

		Wgt.	H. Arm	H. Mom.	V. Arm	CG. Mom.
C_1	.15 (60)(.5) 1/2	0.02	0.40	0.01	19.67	0.39
C_2	.15 (.65)(.50)	0.05	0.93	0.05	19.[illegible]	0.99
C_3	.15 (19.5)(1.25)	3.66	0.625	2.29	9.75	35.69
C_4	.15 (12.96 ◻)	1.95	3.42	6.67	17.9[illegible]	35.[illegible]
C_5	.15 (16.61)(5.9)	14.70	4.20	61.74	8.[illegible]	122.01
C_6	.15 (8.[illegible])(7.17) 1/2	4.63	9.54	44.17	10.87	50.33
C_7	.15 (16.85)(8.0)	22.62	16.53	375.04	4.0	90.45
	ΣV	47.63		ΣM_H 489.97		ΣM_{CG} 334.89

$$\frac{\Sigma M_H}{\Sigma V} = \frac{489.97}{47.63} = 10.29' \qquad \frac{\Sigma M_{CG}}{\Sigma V} = \frac{334.89}{47.63} = 7.03 \qquad e = 13.0 - 10.29 = 2.71'$$

$$f = \frac{47.63}{26.0}\left(1 \pm \frac{6 \times 2.71}{26.0}\right) = 1.835(1 \pm .625) =$$

278 psf @ Heel
686 psf @ Toe

CASE I CONSTRUCTION CONDITION

Conc.		47.63 ↓		489.97		
Wind	[illegible]	.36 ←	14.0	− 5.04		

$\Sigma M = 484.93$

$$\frac{\Sigma M}{\Sigma V} = \frac{484.93}{47.63} = 10.16' > 8.67 < 17.33' \quad O.K. \quad 13.0 - 10.16 = 2.84'$$

$$f = 1.835\left(1 \pm \frac{6 \times 2.84}{26.0}\right) = 1.835(1 \pm .656) = $$

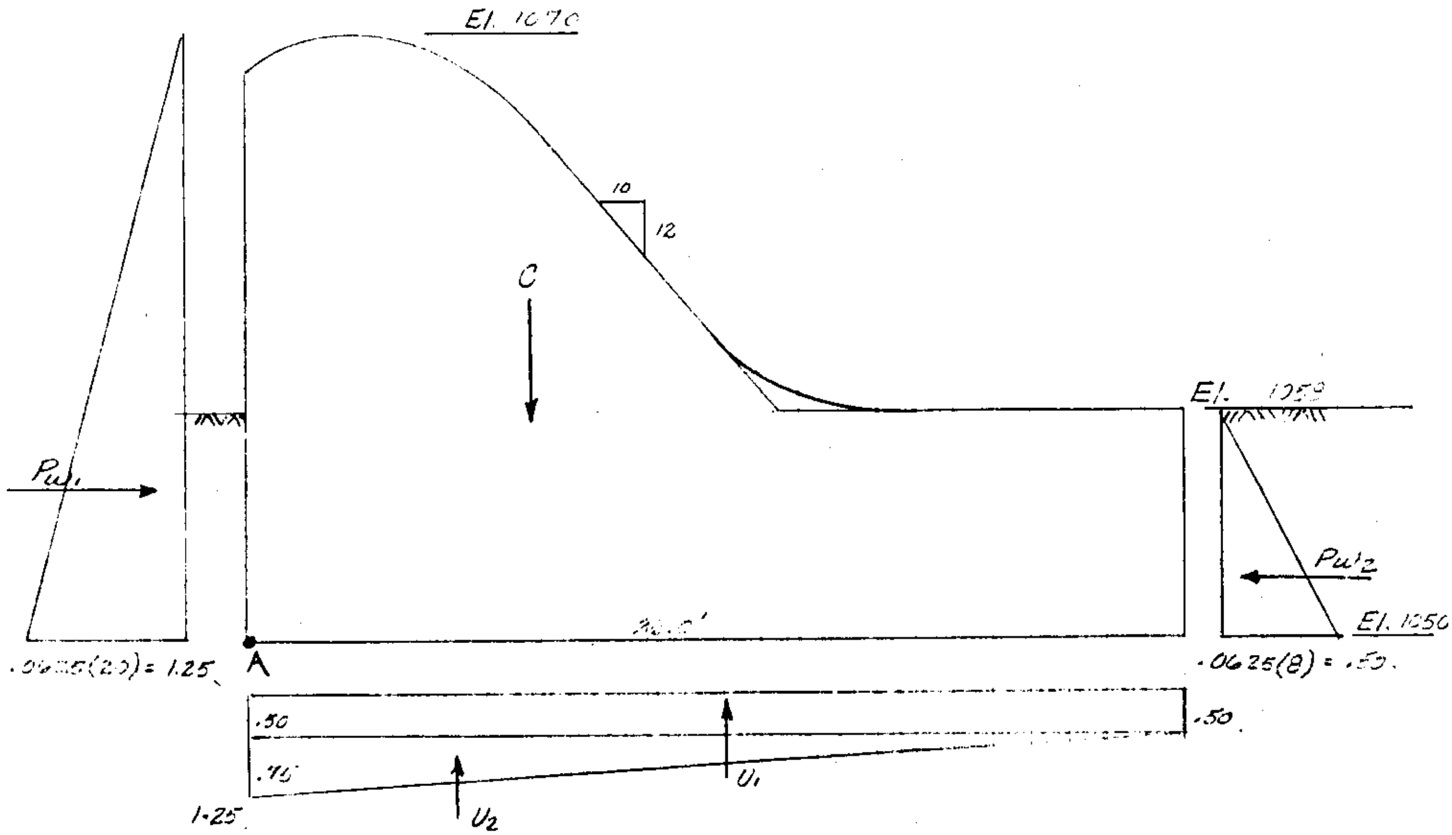
30.[illegible] psf @ Heel
631 psf @ Toe

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Gardner, Mass. Local Protection Project
COMPUTATION Wright's Reservoir Spillway Stability
COMPUTED BY JHM CHECKED BY BJW DATE 20 Sept 62

CASE II Normal Operating Condition



		V	H	Arm	Mom abt A
C		47.63 ↓			489.97
U_1	.50 (26.0)	13.0 ↑		13.0	−169.00
U_2	.75 (26.0) 1/2	9.75 ↑		8.67	−84.50
	ΣV =	24.88 ↓			
P_{w1}	1.25 (20) 1/2		12.5 →	6.67	83.40
P_{w2}	.50 (8) 1/2		2.0 ←	2.67	−5.34
				ΣM =	314.53
			ΣH = 10.5 →		

$$\frac{\Sigma H}{\Sigma V} = \frac{10.5}{24.88} = 0.42 > 0.65 \quad \text{O.K.}$$

$$\frac{\Sigma M}{\Sigma V} = \frac{314.53}{24.88} = 12.63, \text{ within mid } 1/3 \text{ O.K.} \qquad 13.0 - 12.63 = 0.37 \text{ Lt.}$$

$$f = \frac{24.88}{26.0}\left(1 \pm \frac{6 \times 0.37}{26.0}\right) = .958(1 \pm .086) = $$

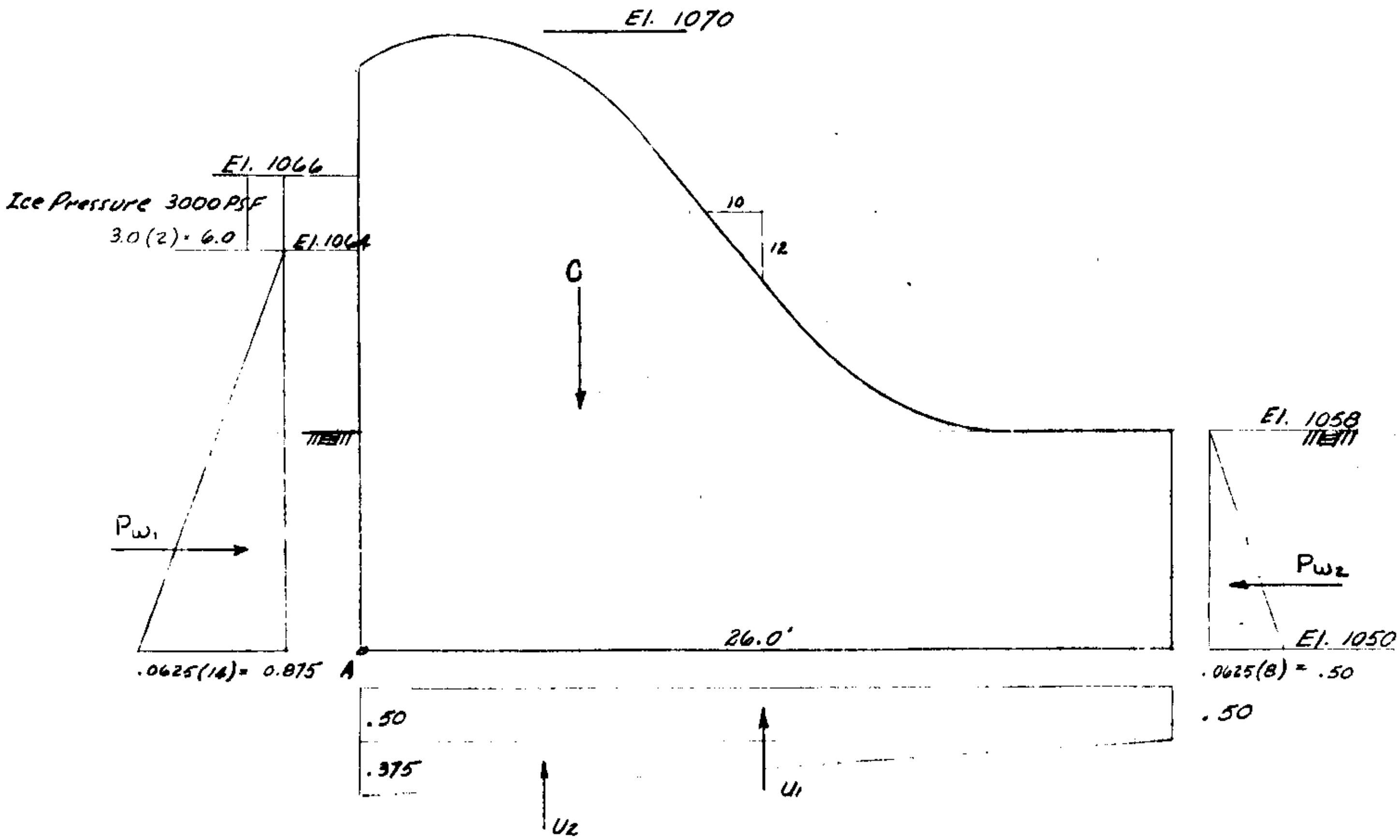
1040. psf @ HEEL
875. psf @ TOE

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Gardner, Mass. Local Protection Project
COMPUTATION Wright's Reservoir Spillway Stability
COMPUTED BY Jmr CHECKED BY BJW DATE Sept 62

CASE III Ice Pressure with Reservoir @ Normal El. 1066



		V	H	Arm	Mom Abt. A
C		47.63 ↓			489.97
U_1	.50(26.0)	13.0 ↑		13.0	-169.00
U_2	.375(26.0)½	4.88 ↑		8.67	-42.30
	$\Sigma V =$	29.75 ↓			
Ice	6.0		6.0 →	15.0	90.00
P_{w_1}	0.875(14)½		6.12 →	4.67	28.60
P_{w_2}	0.50(8)½		2.00 ←	2.67	-5.34
				$\Sigma M =$	391.93

$\Sigma H = 10.12 \rightarrow$

$$\frac{\Sigma H}{\Sigma V} = \frac{10.12}{29.75} = .34 > 0.65 \text{ O.K.}$$

$$\frac{\Sigma M}{\Sigma V} = \frac{391.93}{29.75} = 13.16 \text{ within mid } 1/3 \text{ O.K.}$$

13.16 - 13.0 = 0.16 RT.

$$f = \frac{29.75}{26.0}\left(1 \pm \frac{6 \times .16}{26.0}\right) = 1.144(1 \pm .037) =$$

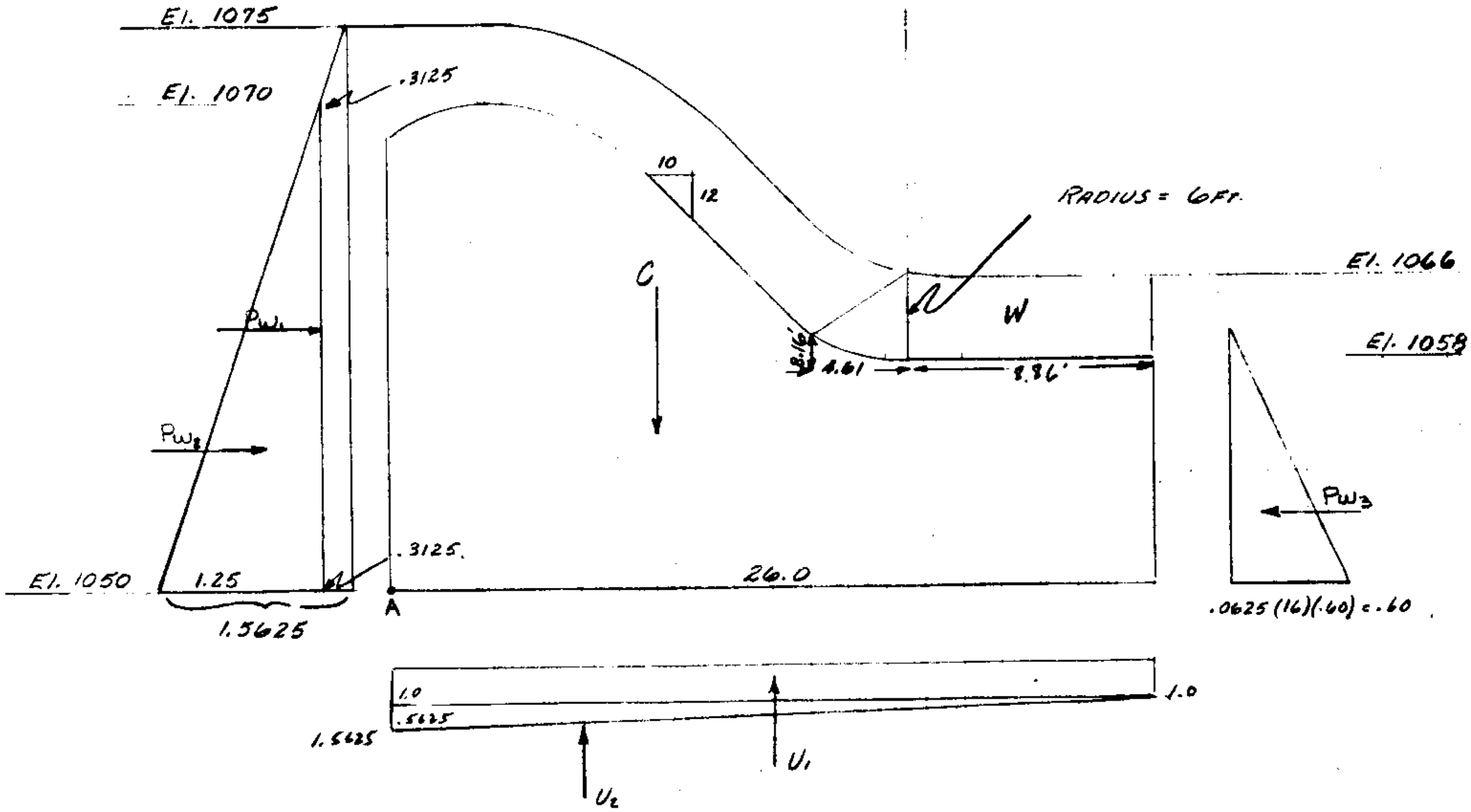
1187 psf @ TOE
1103 psf @ HEEL

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: Gardner, Mass. Local Protection Project
COMPUTATION: Wright's Reservoir Spillway Stability
COMPUTED BY: JMM CHECKED BY: BJW DATE: Sept 62

CASE IV FLOOD DISCHARGE CONDITION



		V	H	Arm	Mom abt. "A"
C		47.63 ↓			489.97
W	.0625(4.61)(8.0) ½	1.15 ↓		15.60	17.94
	.0625(8.86)(8.0)	4.43 ↓		21.57	95.56
U_1	1.0 (26.0)	26.0 ↑		13.0	-338.0
U_2	.5625 (26.0) ½	7.31 ↑		8.67	-63.40
		ΣV = 19.90 ↓			
P_{w_1}	.3125 (20)		6.25 →	10.0	62.50
P_{w_2}	1.25 (20) ½		12.5 →	6.67	83.38
P_{w_3}	.60 (9.6) ½		2.88 ←	3.2	- 9.22
			ΣH = 15.87 →	ΣM =	338.73

$\frac{\Sigma H}{\Sigma V} = \frac{15.87}{19.90} = .80$ High Check shear-friction factor of safety

$S_{S-F} = \frac{.65(19.90) + .5(.075)(26.0)(144)}{15.87} = 9.6 > 4.0$ O.K.

17.02 - 13.0 = 4.02 Rt.

$\frac{\Sigma M}{\Sigma V} = \frac{338.73}{19.90} = 17.02 < 17.32$ within mid ⅓ O.K.

$f = \frac{19.90}{26.0}\left(1 \pm \frac{6 \times 4.02}{26.0}\right) = .765(1 \pm .928)$

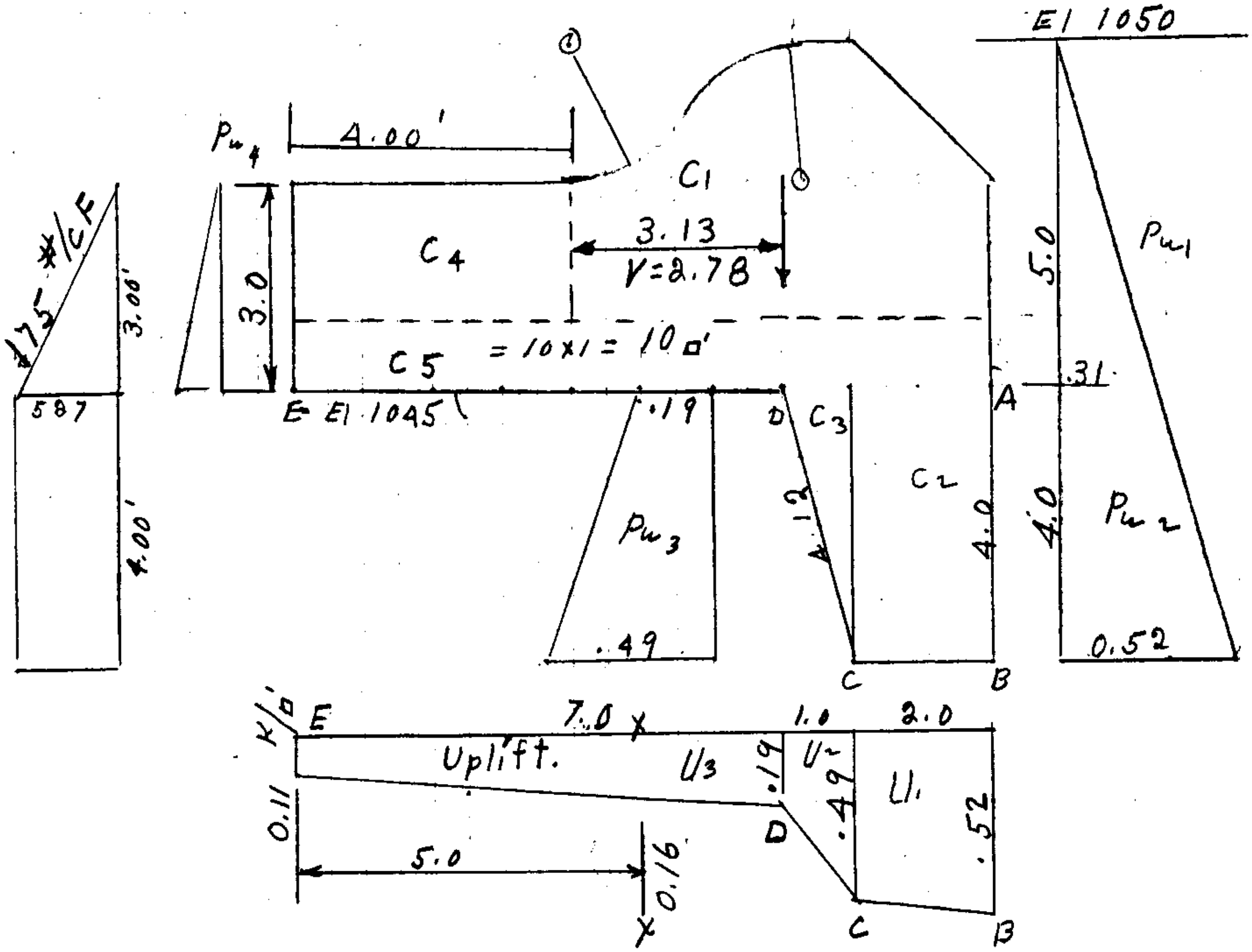
1475 psf TOE
55 psf HEEL

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

PAGE DM-8

SUBJECT GARDNER MASS LOCAL PROTECTION PROJECT
COMPUTATION Mahoney Brook Spillway Stability
COMPUTED BY AP CHECKED BY BMW DATE Sept 62



Diff in Head $1050 - (1045 + 60\% \times 3) = 3.2'$

$\frac{3.2}{17.12} = .187$

		E = 60% 3	1.80 = ~~110~~	0.11 K/□'
7.00 × .187	= 1.31	D = 1.80 + 0.00 + 1.31 = 3.11	= 194	0.19
4.12 × "	= 0.77	C = 3.11 + 4.00 + 0.77 = 7.88	= 492	0.49
2.00 × "	= 0.37	B = 7.88 + 0.00 + 0.37 = 8.25	= 516	0.52
4.00 × "	= 0.75	A = 8.25 − 4.00 + 0.75 = 5.00	= 313	0.31
17.12	3.20			

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U.S. ARMY

PAGE DM-9

SUBJECT GARDNER MASS LOCAL PROTECTION PROJECT
COMPUTATION Mahoney Brook Spillway Stability
COMPUTED BY R CHECKED BY BTW DATE Sept 1962

		→ KIP	← KIP	↑ KIP	↓ KIP	Arm	Mo	MR
C1					2.78	7.13		19.82
C2	4 x 2.0 x 0.15				1.20	9.00		10.80
C3	4 x 0.5 x 0.15				0.30	7.67		2.30
C4	4 x 2.0 x 0.15				1.20	2.00		2.40
C5	10 x 1.0 x 0.15				1.50	5.00		7.50
Case I Conc					6.98	6.13		42.82
Pw 1	0.31 x 2.5		0.78			1.67	1.30	
Pw 2	0.31 x 2.0		0.61			-1.33		0.81
Pw 2	0.52 x 2.0		1.04			-2.67		2.78
Pw 3	0.49 x 2.0	0.98				-2.67	2.62	
Pw 3	0.19 x 2.0	0.38				-1.33	0.51	
Pw 4	0.11 x 1.5	0.17				+1.00		0.17
U 1	0.52 x 1.0			0.52		9.33	4.85	
U 1	0.49 x 1.0			0.49		8.67	4.25	
U 2	0.49 x 0.5			0.25		7.67	1.92	
U 2	0.19 x 0.5			0.10		7.33	0.73	
U 3	0.19 x 3.5			0.66		4.67	3.08	
U 3	0.11 x 3.5			0.38		2.33	0.89	
Case II		1.53	2.43	2.40	6.98 4.58	5.77	20.15	46.58 (26.43)
Ice			2.00			4.67	9.34	
	1.5 x 527	0.79				1.00		0.79
	4.0 x 527	2.11				-2.00	4.22	
Case III		4.43	4.43	2.40	6.98 4.58	2.98	33.71	47.37 13.66

L/4 = 2.25

Conc Only
$\frac{P}{L}(1 \pm \frac{6e}{L}) = 698(1 \pm .68) = 1173$ & 223 CASE I (B, E)

WS 1050 No Ice
$\frac{P}{L}(1 \pm \frac{6e}{L}) = 458(1 \pm .46) = 669$ & 247 CASE II

Ice 2000# $\frac{458}{2.98} \times \frac{2}{3} = 1025$ PSF CASE III

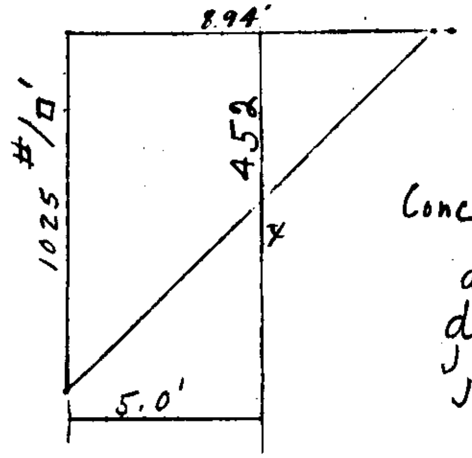
Uplift Sec X-X
110 x 2.5 = 275. x 3.33 = 915.1#
160 x 2.5 = 400. x 1.67 = 668.1#
675.# 1583

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT GARDNER MASS LOCAL PROTECTION PROJECT
COMPUTATION Mahoney Brook Spillway Stability
COMPUTED BY AP CHECKED BY BJW DATE Sept 62

CASE III ICE at NORMAL POOL ELEV. – Ice, 2000#



	↓		K.F Mom	
2.5×1.025	2.56	3.33	8.52	
2.5×.452	1.13	1.67	1.89	
Uplift	.68	–	1.58	
	4.37		11.99	
Conc 5×0.45	2.25	2.5	5.63	
	-2.12		6.36	$A_s = \frac{M}{ad}$

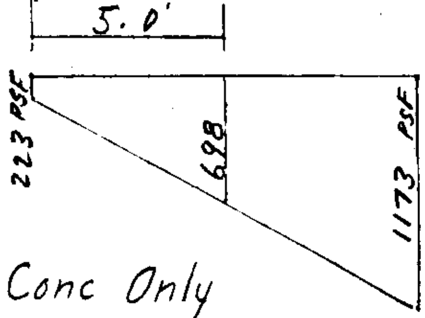
$a = 2.0$
$d = 32$ $\quad v = \frac{2120}{28.8 \times 12} = 61\ \#/\square'$
$J = .9$
$Jd = 28.8$

$A_s = \frac{6.36}{64} = 0.10\ \square''$

U = 210 Top Bar
300 Bot "

$\Sigma_0 = \frac{V}{UJd} = \frac{2120}{300 \times 28.8} = 0.25''$

CASE I CONSTRUCTION CONDITION

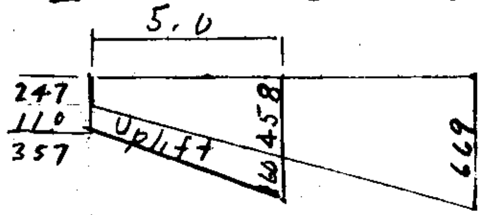


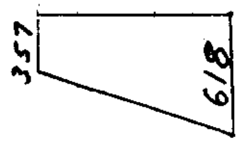
Conc Only

2.5 × 698 = 1745	1.67	2914
2.5 × 223 = 558	3.33	1858
2303		4772

Will not Affect Design

CASE II NORMAL OPERATING COND.



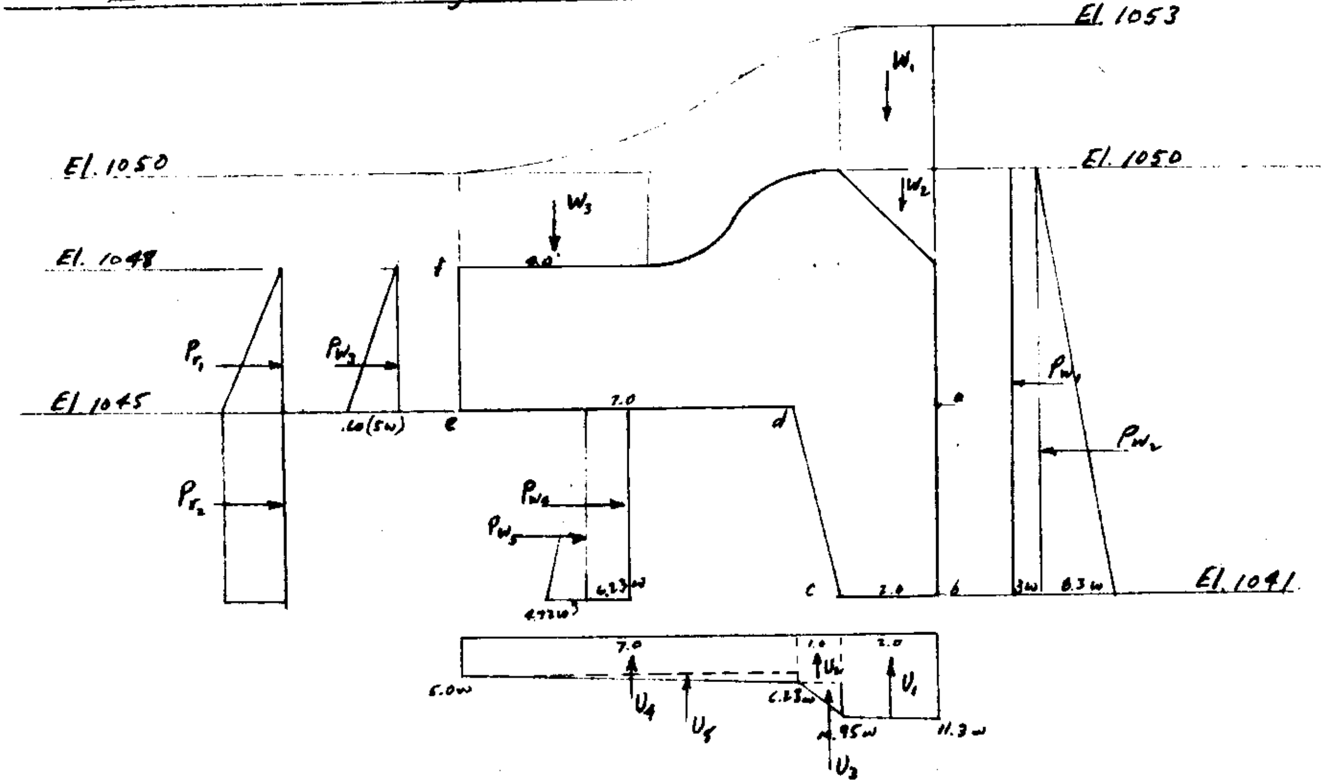


2.5 × 618 = 1545	1.67	2580
2.5 × 357 = 892	3.33	2972
2437		5552
Conc – 2250		–5630

d = 32 a = 1.44

SUBJECT Gardner, Mass Local Protection Project
COMPUTATION Mahoney Brook Spillway Stability
COMPUTED BY BW CHECKED BY J.W.F. DATE Sept. 62

CASE IV Flood Discharge Condition



Creep distance abcde = 4.0 + 2.0 + 4.12 + 7.0 = 17.12'

Creep Ratio = $\frac{17.12}{3.0}$ = 5.7 > 4.0 O.K. ✓

	Pos. Pot.		Seep. Pot.		
a	5.0	+	3.0	=	8.0
b	9.0	+	$\frac{13.12}{17.12}(3.0)$	=	11.3
c	9.0	+	$\frac{11.13}{17.12}(3.0)$	=	10.95
d	6.0	+	$\frac{2.0}{17.12}(3.0)$	=	6.23
e	5.0	+	0	=	5.0

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Gardner, Mass Local Protection Project
COMPUTATION Mahoney Brook Spillway Stability
COMPUTED BY BW CHECKED BY JWF DATE Sept. 62

CASE IV contd.

		↓	↑	→	←	arm	Mabt e
C		6.98				6.13	42.82
W_1	.0625 × 3.0 × 2.0	.37				9.0	3.38
W_2	.0625 × 2.0 × 2.0 × ½	.12				9.33	1.17
W_3	.0625 × 2.0 × 4.0	.50				2.0	1.0
U_1	.0625 × 11.12 × 2.0		1.39			9.0	− 12.51
U_2	.0625 × 6.23 × 1.0		.39			7.5	− 2.92
U_3	.0625 × 4.72 × 1.0 × ½		.15			7.67	− 1.13
U_4	.0625 × 5.0 × 7.0		2.19			3.5	− 7.66
U_5	.0625 × 1.23 × 7.0 × ½		.27			4.67	− 1.26
P_{w_1}	.0625 × 3.0 × 9.0				1.69	0.5	− .84
P_{w_2}	.0625 × 8.3 × 9.0 × ½				2.34	−1.0	2.34
P_{w_3}	.0625 × 3.0 × 3.0 × ½			.28		1.0	.28
P_{w_4}	.0625 × 6.23 × 4.0			1.56		−2.0	− 3.12
P_{w_5}	.0625 × 4.72 × 9.0 × ½			.59		−2.67	− 1.58
		7.97	4.39	2.43	4.03		19.97
		3.58			1.60		
	$\Sigma H = P_{r_1} + P_{r_2} = 1.60$ $5.5x = 1.60 \quad x = .29$						
P_{r_1}	.29 × 3.0 × ½			.44		1.0	.44
P_{r_2}	.29 × 4.0			1.16		−2.0	− 2.32
	ΣV	3.58				ΣM	18.09

$$\frac{\Sigma M}{\Sigma V} = \frac{18.09}{3.58} = 5.05' \quad \text{o.k. Within Mid } 1/3 \qquad e = 5.05 - 5.0 = 0.05' \text{ Rt.}$$

$$f = \frac{3.58}{10.0}\left(1 \pm \frac{6 \times .05}{10}\right) = 0.358\,(1 \pm .03) = 369 \text{ PSF @ Heel}$$
$$347 \text{ PSF @ Toe}$$

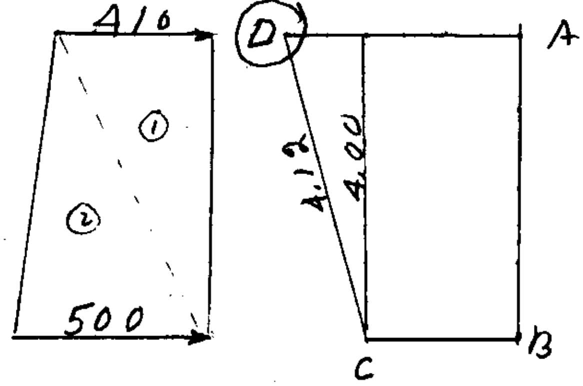
SUBJECT GARDNER MASS LOCAL PROTECTION PROJECT

COMPUTATION Mahoney Brook Spillway

COMPUTED BY A CHECKED BY BNW DATE Sept 62

DESIGN OF KEY

2000# Ice



A
527.
190.
717
310.
~~407~~. 410.

B
527.
490.
1017.
520.
~~497~~. 500.

① $.82 \times 1.33 = 1.1^{1K}$
② $1.00 \times 2.67 = 2.7$
$\Sigma H = 1.82$ $\quad \Sigma M = 3.8^{1K}$

$A_s = \frac{3.8}{64} \times \frac{4.12}{4} = 0.06''$

Use Min. #6 @ 12"

NED FORM 223
27 Sept 49

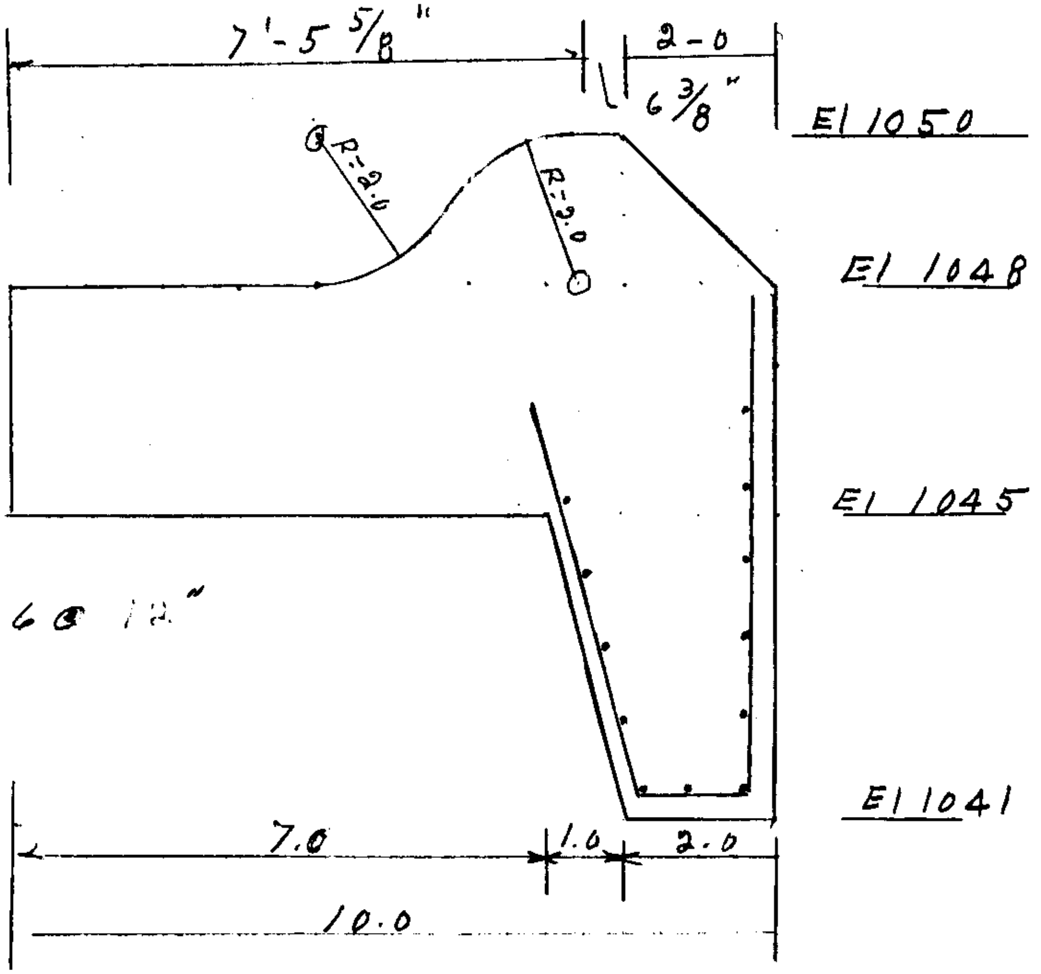
NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U. S. ARMY

SUBJECT GARDNER MASS Local Prot Proj.

COMPUTATION Mahoney Brook Spillway

COMPUTED BY AP CHECKED BY RAW DATE Sept 62

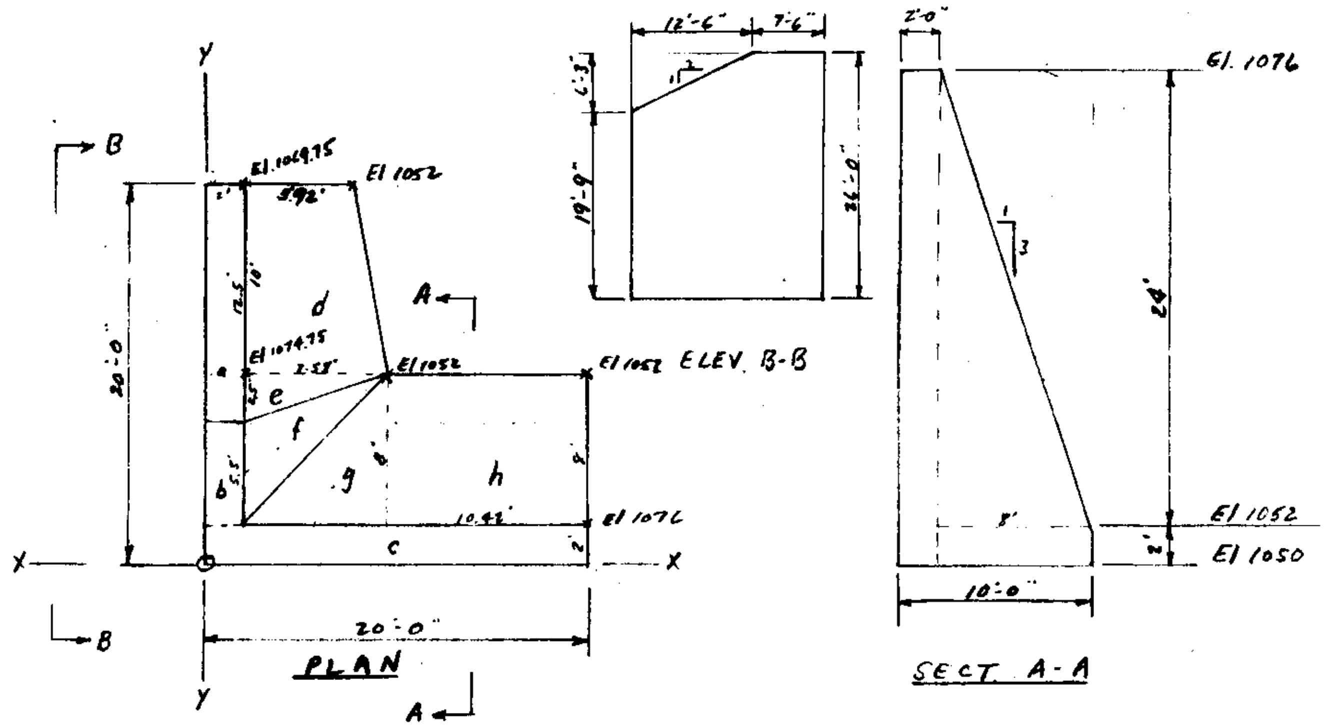


All bars #6 @ 12"

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Gardner Mass Local Protection Project
COMPUTATION Wright's Reservoir Spillway Abutment
COMPUTED BY BPW CHECKED BY JWF DATE OCT. 1962



Concrete Only

			X-ARM	M_{Y-Y}	Y-ARM	M_{X-X}
a	$.15 \times 2.0 \times 12.5 \times 6.25 \times \frac{1}{2}$	11.72	1.0	11.7	11.67	136.9
a'	$.15 \times 2.0 \times 12.5 \times 19.75$	74.1	1.0	74.1	13.75	1019.
b	$.15 \times 2.0 \times 5.5 \times 26.0$	42.9	1.0	42.9	4.75	203.6
c	$.15 \times 20.0 \times 2.0 \times 26.0$	156.0	10.0	1560.	1.0	156.
d	$.15 \times \frac{10.0}{3}\left(\frac{17.75 \times 5.92}{2} + \frac{22.75 \times 7.58}{2} + \sqrt{(52.5)(86.2)}\right)$	103.0	4.27	440.	14.59	1502.
d'	$.15 \times 10.0 \times \frac{5.92+7.58}{2} \times 2.0$	20.2	5.38	109.	15.0	303.5
e	$.15 \times \frac{7.58}{3} \times \frac{22.75+24.0}{2} \times 2.5$	22.2	3.89	86.	9.06	201.
f	$.15 \times \frac{7.58}{3} \times 5.5 \times 24.0$	50.0	3.89	194.6	6.06	303.
g	$.15 \times \frac{8.0}{3} \times 7.58 \times 24.0$	72.8	6.74	490.	4.0	291.
h	$.15 \times 24.0 \times 8.0 \times \frac{1}{2} \times 10.42$	150.	14.79	2220.	4.67	701.
e'f'g'h'	$.15 \times 18.0 \times 8.0 \times 2.0$	43.2	11.0	475.	6.0	259.
		746.1		57033		5076.0

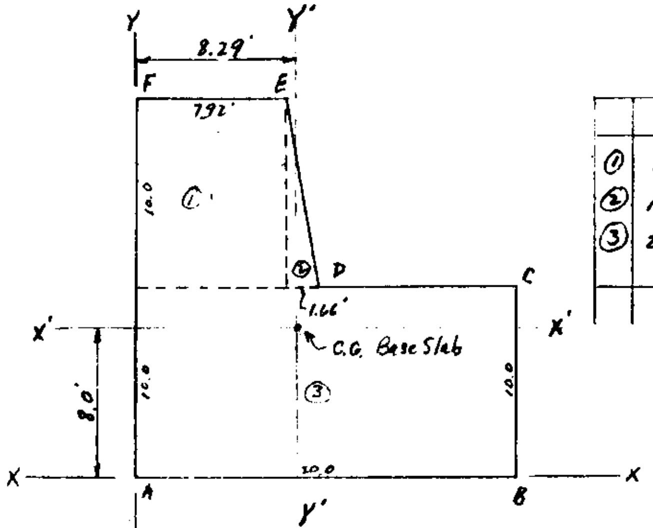
$$\bar{x} = \frac{5703.3}{746.1} = 7.65'$$

$$\bar{y} = \frac{5076.0}{746.1} = 6.81'$$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION
CORPS OF ENGINEERS, U. S. ARMY

SUBJECT Gardner Mass Local Protection Project
COMPUTATION Wright's Reservoir Spillway Abutment
COMPUTED BY BFW CHECKED BY JWF DATE OCT 1962



PLAN OF BASE SLAB

		AREA	x-ARM	M_{Y-Y}	y-ARM	M_{X-X}
①	7.92 x 10.0	79.2	3.96	314	15.0	1188
②	1.66 x 10.0 x ½	8.3	8.47	70	13.33	111
③	20.0 x 10.0	200.0	10.0	2000	5.0	1000
		287.5	8.29	2384	8.0	2299

	AREA - SF	d_x	$\bar{d}_x^2$	d_y	$\bar{d}_y^2$	$\bar{I}_{x-y}$	$A d_x d_y$	$I_{xy} = \bar{I}_{x-y} + A d_x d_y$
①	79.2	-4.33	18.75	7.0	49.0	0	-2400	-2400
②	8.3	0.18	.03	5.33	28.41	3.83	7.95	12
③	200.0	1.71	2.92	-3.0	9.0	0	-1026	-1026
								-3414

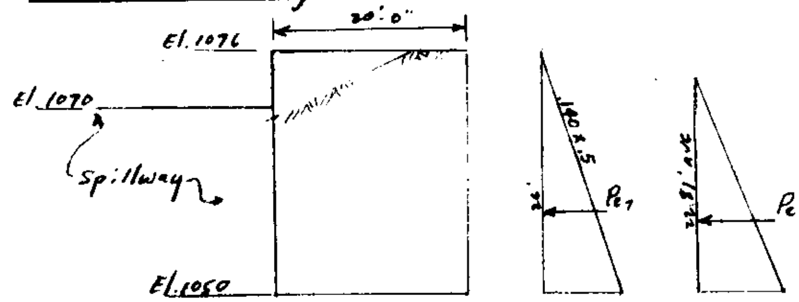
	I_{cgx}	I_{cgy}	$A\bar{d}_x^2$	$A\bar{d}_y^2$	$I_{cgx} + A\bar{d}_y^2$ I_{x-x}	$I_{cgy} + A\bar{d}_x^2$ I_{y-y}
①	660	414	1485	3881	4541	1899
②	46	1	0	236	282	1
③	1667	6667	584	1800	3467	7251
	2373	7082	2069	5917	8290	9151

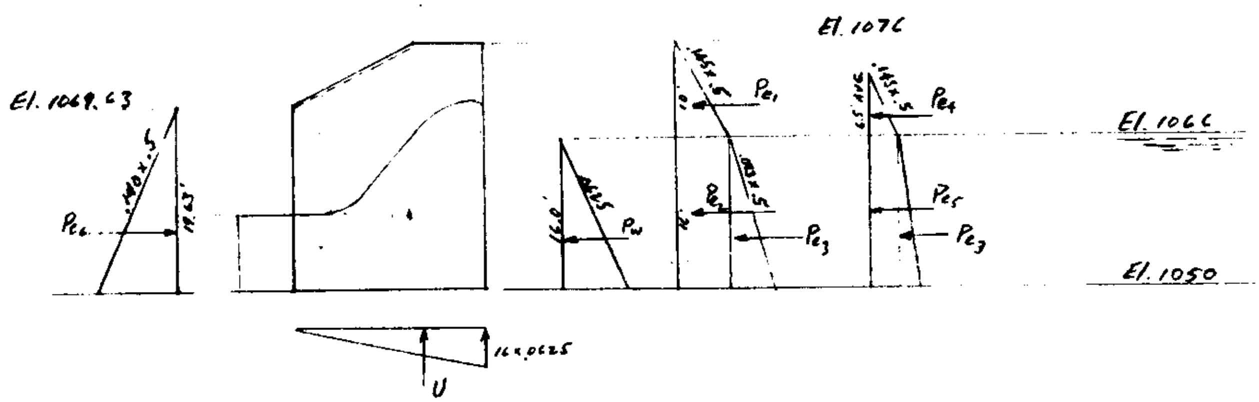
SUBJECT Gardner, Mass. Local Protection Project
COMPUTATION Wright's Reservoir Spillway Abutment
COMPUTED BY BJM CHECKED BY JWF DATE OCT 1962

CASE I No water over spillway, Normal Pool El. 1066

Lateral Load Diags.

At rest pressures
K = 0.5





NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U. S. ARMY

PAGE DM-18

SUBJECT Gardner, Mass Local Protection Project

COMPUTATION Wright's Reservoir Spillway Abutment

COMPUTED BY BJW CHECKED BY JWF DATE OCT 1962

CASE I contd

		↓	↑	→	←	X-ARM	M_{Y-Y}	Y-ARM	M_{X-X}
Conc.		746.1					5703.3		5076.0
Earth									
d	$\frac{14}{15}$ (103.0)	96.0				6.48	623.	14.59	1402.
	.14 × 8.0 × 24.0 × $\frac{1}{2}$ × 18.0	242.				11.0	2658	7.33	1770
Lateral Pressures						Z ARM		Z-ARM	
P_w	.0625 × $\overline{16.0}^2$ × $\frac{1}{2}$ × 20.0				A 160.			5.33	853
P_{e1}	.145 × .5 × $\overline{10}^2$ × $\frac{1}{2}$ × 6.0				A 21.7			19.80	430
P_{e2}	.145 × .5 × 10 × 16 × 6.0				A 69.5			8.0	556
P_{e3}	.083 × .5 × $\overline{16}^2$ × $\frac{1}{2}$ × 20.0				A 106.2			6.07	645
P_{e4}	.145 × .5 × $\overline{6.5}^2$ × $\frac{1}{2}$ × 14.0				A 21.4			18.17	389
P_{e5}	.145 × .5 × 6.5 × 16 × 14.0				A 105.5			8.0	844
P_{e6}	.140 × .5 × $\overline{19.63}^2$ × $\frac{1}{2}$ × 20.0			A 270				7.46	− 2015
P_{e7}	.140 × .5 × $\overline{26}^2$ × $\frac{1}{2}$ × 7.25				B 172	9.88	− 1700		
P_{e8}	.140 × .5 × $\overline{22.51}^2$ × $\frac{1}{2}$ × 12.75				B 232.5	8.67	− 2020		
uplift	.0625 × 16 × $\frac{7.92}{2}$ × 20		79.2			3.96	− 314	6.67	− 528
	.0625 × 8 × 10 × $\frac{1}{2}$ × 1.66/2		2.1			8.53	− 18	12.5	− 126
	.0625 × 8 × 10 × 12.08		60.4			13.75	− 830	5.0	− 302
	.0625 × 8 × 10 × 12.08 × ½		30.2			13.75	− 415	3.33	− 101
	Σ	1084.1	171.9	(A) 270	(A) 484.3 (B) 404.5		3687		8993
	Σ	912.2^k			(A) 214.3^k		3687$^{'k}$		8993$^{'k}$
	Spillway reaction			404.5^k	(B) 404.5^k	8'	+3236 6923$^{'k}$		

$$\bar{x} = \frac{6{,}923}{912.2} = 7.59'$$

$$\bar{y} = \frac{8993}{912.2} = 9.85'$$

SUBJECT Gardner, Mass. Local Protection Project
COMPUTATION Wright's Reservoir Spillway Abutment
COMPUTED BY BPM CHECKED BY JWF DATE OCT 1962

CASE I

Base Bearing Pressure

Constants $P = 912.2^{k}$ $\quad e_x = -0.70'$ $\quad M_{y-y} = Pe_x = -638'^{k}$
$A = 287.5^{\square'}$ $\quad e_y = 1.85'$ $\quad M_{x-x} = Pe_y = 1{,}690'^{k}$
$P/A = 3.18^{k/\square'}$

$I_{x-x} = 8290$ $\quad M_{y-y} I_{x-x} = -5{,}289{,}000$
$I_{y-y} = 9151$ $\quad M_{x-x} I_{y-y} = 15{,}500{,}000$
$I_{x-y} = -3414$ $\quad I^2_{x-y} = 11{,}655{,}000$

$I_{x-x} I_{y-y} = 75{,}862{,}000$ $\quad I_{x-x} I_{y-y} - I^2_{x-y} = 64{,}207{,}000$

$M_{x-x} I_{x-y} = -5{,}770{,}000$ $\quad M_{y-y} I_{x-x} - M_{x-x} I_{x-y} = 481{,}000$
$M_{y-y} I_{x-y} = 2{,}178{,}000$ $\quad M_{x-x} I_{y-y} - M_{y-y} I_{x-y} = 13{,}322{,}000$

$$M = \frac{M_{y-y} I_{x-x} - M_{x-x} I_{x-y}}{I_{x-x} I_{y-y} - I^2_{x-y}} = \frac{481{,}000}{64{,}207{,}000} = 0.00749$$

$$N = \frac{M_{x-x} I_{y-y} - M_{y-y} I_{x-y}}{I_{x-x} I_{y-y} - I^2_{x-y}} = \frac{13{,}322{,}000}{64{,}207{,}000} = 0.207$$

$$f = \frac{P}{A} \pm x_o M \pm y_o N$$

$f_A = 3.18 - 8.29(.00749) - 8(0.207) = 3.18 - 0.06 - 1.66 = 1.46^{KSF}$

$f_B = 3.18 + 11.71$ " -8 " $= 3.18 + 0.09 - 1.66 = 1.61^{KSF}$

$f_C = 3.18 + 11.71$ " $+2$ " $= 3.18 + 0.09 + 0.41 = 3.68^{KSF}$

$f_D = 3.18 + 1.29$ " $+2$ " $= 3.18 + 0.01 + 0.41 = 3.60^{KSF}$

$f_E = 3.18 - 0.37$ " $+12$ " $= 3.18 - 0.00 + 2.48 = 5.66^{KSF}$

$f_F = 3.18 - 8.29$ " $+12$ " $= 3.18 - 0.06 + 2.48 = 5.60^{KSF}$

NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U.S. ARMY

PAGE DM-20

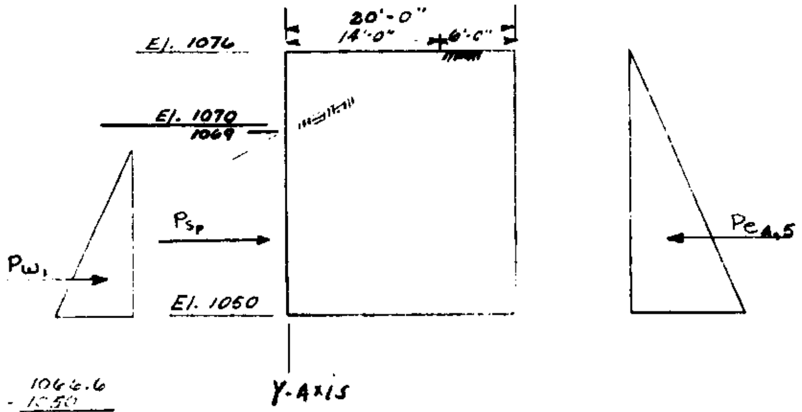
SUBJECT: GARDNER, MASS. LOCAL PROTECTION PROJECT

COMPUTATION: WRIGHT'S RESERVOIR SPILLWAY ABUTMENT

COMPUTED BY: JAM   CHECKED BY: BMW   DATE: OCT 62

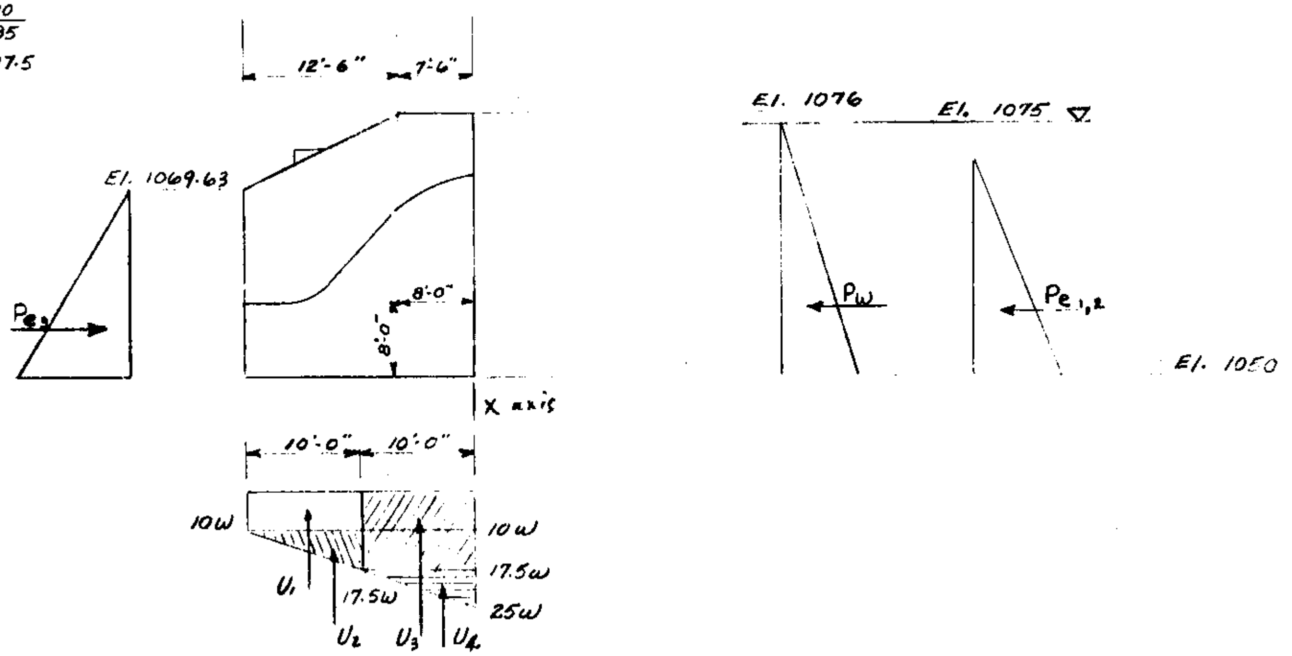
CASE II - WATER OVER spillway @ FLOOD Discharge Condition, Assume FULL WATER HEAD Down to .6 of Effective tailwater. Use saturated soil behind walls with no water pressure. Figure Uplift over base from full head to .6 tailwater. At rest earth pressure K = 0.5

LATERAL LOAD DIAGRAMS



1066.6
- 1050
16.6
× .6
9.96 or 10.0

25
10
2 | 35
17.5



NED FORM 223
27 Sept 49

NEW ENGLAND DIVISION

CORPS OF ENGINEERS, U. S. ARMY

SUBJECT: GARDNER, MASS. LOCAL PROTECTION PROJECT

COMPUTATION: WRIGHT'S RESERVOIR SPILLWAY ABUTMENT

COMPUTED BY: JFM CHECKED BY: BJW DATE: OCT 62

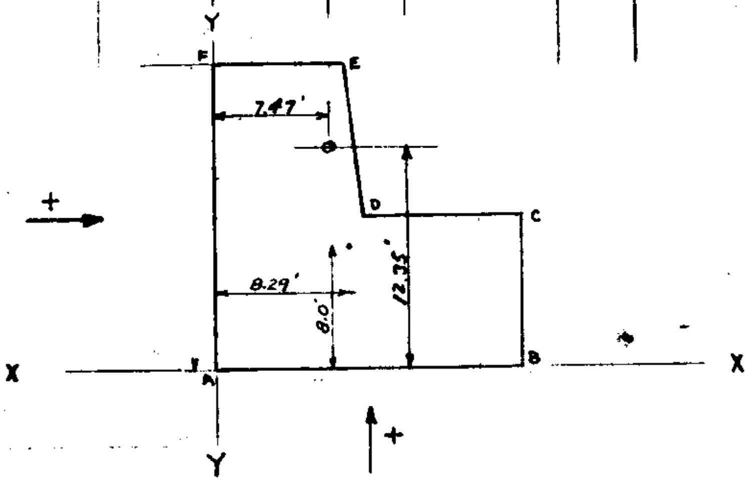
CASE II STABILITY ANALYSIS

1066.6
1050.0
16.6
×.6
9.96 or 10.0
25
10
2|35
17.5

		↓	↑	→	←	X-Arm	M_{Y-Y}	Y-Arm	M_{X-X}
Earth		338.0					3281.0		3172.0
Conc.		746.1					5703.3		5076.0
Pressure								Z Arm	
P_w	25.0×25.0×.0625×½×20.0				A 390.8			8.33	3255.4
P_{e1}	26.0×26.0×.083×½×6.0×.50				A 84.2			8.67	730.0
P_{e2}	$\overline{22.5}^2$×½×.083×14.0×.50				A 147.0			7.50	1102.5
P_{e3}	$\overline{19.63}^2$×½×20.0×.144×.50			A 277.0				6.54	-1811.6
Pressure						Z Arm			
P_{w1}	$\overline{17.5}^2$×½×20.0×.0625			B 191.6		5.83	1117.0		
P_{e4}	$\overline{26.0}^2$×½×7.5×.144×.50				B 182.3	8.67	-1580.5		
P_{e5}	$\overline{22.8}^2$×½×12.5×.144×.50				B 234.0	7.60	-1778.0		
P_w	182.3+234.0−191.6			B 224.7		8.00	1797.6		
Uplift									
U_1	10×.0625×10×8.75		54.7			4.38	-239.6	15.0	-820.5
U_2	7.5×.0625×10×8.75×½		20.5			4.38	-89.8	13.3	-272.7
U_3	17.5×.0625×10×20		218.8			10.0	-2188.0	5.0	-1094.0
U_4	7.5×.0625×10×20×½		46.9			10.0	-469.0	3.33	-156.2
Σ		1084.1	340.9	A 277.0 B 416.3	A 622.0 B 416.3		5554.0		9180.9
Σ		743.2			A 345.0 B 0		5554.0		9180.9

$$\bar{x} = \frac{5554.0}{743.2} = 7.47'$$

$$\bar{y} = \frac{9180.9}{743.2} = 12.35'$$



SUBJECT GARDNER, MASS. LOCAL PROTECTION PROJECT
COMPUTATION WRIGHT'S RESERVOIR SPILLWAY ABUTMENT
COMPUTED BY JMM CHECKED BY BJW DATE OCT 62

CASE II
BASE BEARING PRESSURES

CONSTANTS

$P = 743.2$ $\quad e_x = -0.82$ $\quad M_{y\text{-}y} = Pe_x = -609'^{K}$
$A = 287.5^{\square'}$ $\quad e_y = 4.35$ $\quad M_{x\text{-}x} = Pe_y = 3233'^{K}$
$P/A = 2.59^{K/\square'}$

$I_{x\text{-}x} = 8290$ $\quad M_{y\text{-}y} I_{x\text{-}x} = -5{,}049{,}000$
$I_{y\text{-}y} = 9151$ $\quad M_{x\text{-}x} I_{y\text{-}y} = 29{,}585{,}000$
$I_{x\text{-}y} = -3414$ $\quad I^2_{x\text{-}y} = 11{,}655{,}000$

$I_{x\text{-}x} I_{y\text{-}y} = 75{,}862{,}000$ $\quad I_{x\text{-}x} I_{y\text{-}y} - I^2_{x\text{-}y} = 64{,}207{,}000$

$M_{x\text{-}x} I_{x\text{-}y} = -11{,}037{,}000$ $\quad M_{y\text{-}y} I_{x\text{-}x} - M_{x\text{-}x} I_{x\text{-}y} = 5{,}988{,}000$
$M_{y\text{-}y} I_{x\text{-}y} = 2{,}079{,}000$ $\quad M_{x\text{-}x} I_{y\text{-}y} - M_{y\text{-}y} I_{x\text{-}y} = 27{,}506{,}000$

$$M = \frac{M_{y\text{-}y} I_{x\text{-}x} - M_{x\text{-}x} I_{x\text{-}y}}{I_{x\text{-}x} I_{y\text{-}y} - I^2_{x\text{-}y}} = \frac{5{,}988{,}000}{64{,}207{,}000} = 0.0933$$

$$N = \frac{M_{x\text{-}x} I_{y\text{-}y} - M_{y\text{-}y} I_{x\text{-}y}}{I_{x\text{-}x} I_{y\text{-}y} - I^2_{x\text{-}y}} = \frac{27{,}506{,}000}{64{,}207{,}000} = 0.4284$$

$$f = \frac{P}{A} \pm x_o M \pm y_o N$$

$f_A = 2.59 - 8.29\ (0.0933) - 8.0\ (0.4284) = 2.59 - .77 - 3.43 = -1.61$ KSF

$f_B = 2.59 + 11.71$ " -8.0 " $= 2.59 + 1.09 - 3.43 = 0.25$ KSF

$f_C = 2.59 + 11.71$ " $+2.0$ " $= 2.59 + 1.09 + .86 = 4.54$ KSF

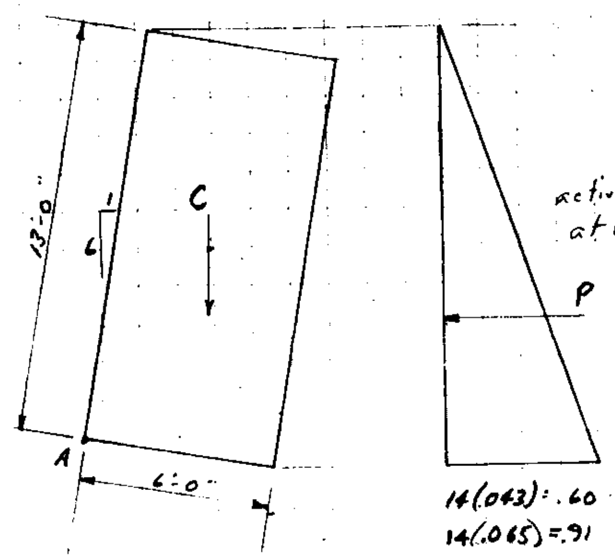
$f_D = 2.59 + 1.29$ " $+2.0$ " $= 2.59 + .12 + .86 = 3.57$ KSF

$f_E = 2.59 - 0.37$ " $+12.0$ " $= 2.59 - .03 + 5.14 = 7.70$ KSF

$f_F = 2.59 - 8.29$ " $+12.0$ " $= 2.59 - .77 + 5.14 = 6.96$ KSF

SUBJECT GARDNER MASS LOCAL PROT PROJ
COMPUTATION CRIB WALL
COMPUTED BY BJW CHECKED BY JWF DATE Sept. 1962

Conc. Crib & Fill,
Use wt of 140 PCF



Earth δ_{dry} : 130 PCF

Eq Fl. Press =
active press. - 0.33(130) = 43 PSF
at rest - .5(130) = 65 PSF

14(.043) = .60
14(.065) = .91

			ARM	M abt A
C	13.0 (6.0)(.140)	10.92	4.04	44.2
P	.60 (14.0) 1/2 .91 (14.0) 1/2	4.20 6.37	3.67 3.67	15.4 23.4
			ΣM	28.8 20.8

$\frac{\Sigma M}{\Sigma V} = \frac{28.8}{10.92} = 2.63'$ o.k. Within Mid 1/3 $\quad e = 3.0 - 2.63 = 0.37'$ LT.

$\frac{\Sigma H}{\Sigma V} = \frac{4.20}{10.92} = 0.38$

$f = \frac{10.92}{6.0}\left(1 \pm \frac{6 \times 0.37}{6.0}\right) = 1.82(1 \pm 0.37) =$ 2490 PSF max
1146 PSF min

Using At Rest Earth Pressure:-

$\frac{\Sigma M}{\Sigma V} = \frac{20.8}{10.92} = 1.90'$ o.k. Within Mid 1/2

$\frac{\Sigma H}{\Sigma V} = \frac{6.37}{10.92} = 0.58$

$f = \frac{2}{3} \times \frac{10.92}{1.90} = 3830$ PSF

TEST DATA SUMMARY

EXPL. NO.	TOP ELEV. FT.	SAMPLE NO.	DEPTH FT.	SOIL SYMBOL	MECHANICAL ANALYSIS GRAVEL %	SAND %	FINES %	D_{10} MM.	ATT. LIMITS LL	PL	SPECIFIC GRAVITY	NAT WATER CONTENT % DRY WT TOTAL	-NO 4	COMPACTION DATA STND AASHO OPT. WATER % DRY WT	MAX DRY DENS. LBS/CU FT	* PVD LBS/CU FT	NAT. DRY DENSITY LBS/CU FT TOTAL	-NO 4	OTHER TESTS SHEAR	CONSOL	PERM.
FD-1	[illegible]	J-1	0.8- 5.0	GM	53	29	18														
FD-2	1068.4	J-1	1.0- 5.0	SM	19	47	34														
FD-3	[illegible]	J-1	10.0-12.5	SM	13	50	37														
		J-5	10.0-12.5	GM	51	21	28														
FD-4	[illegible]	J-3	5.0- 8.0	GM	48	36	16														
FD-5	[illegible]	J-2	5.0-10.0	GP-GM	51	42	7														
		J-4	11.0-12.2	SM	22	53	25														
FD-6	1071.7	J-3	5.0-10.0	SM	31	48	21														
		J-4	10.0-13.6	GM	47	37	16														

* PROVIDENCE VIBRATED DENSITY TEST.

GARDNER, MASS.

PLATE NO. S-1